NAVAL FACILITIES ENGINEERING SERVICE CENTER
Port Hueneme, California 93043-4370

# TECHNICAL REPORT

## TR-2103-SHR

# SEISMIC CRITERIA FOR CALIFORNIA MARINE OIL TERMINALS

# VOLUME 2

by
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With Appendices by
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The Navy and the California State Lands Commission entered into a Cooperative Research and Development Agreement for the development of seismic design criteria for waterfront construction. Both organizations face similar problems in the safe design of facilities and the need for a design guide. The California State Lands Commission (CSLC) has oversight of over sixty marine oil terminals, some of which are over eighty years old and built to unknown standards. Typically, they were built to resist minor earthquake intensity. New earthquake hazard information from recent events such as Loma Prieta (1989) and Northridge (1994) indicates that much higher intensities are possible. It is prudent that these facilities be evaluated and unsafe deficiencies corrected.

This document develops and expands on work that was begun by the Naval Facilities Engineering Service Center to provide seismic design criteria for waterfront construction. This report presents criteria that are intended to define a minimum level of acceptable performance for marine oil terminals and seven chapters and three appendices of technical supporting material. The development of the criteria recognized the need to protect the environment from oil spills, the need to provide for the transfer of required natural resources into the State, and the economics of operating a commercial facility in a competitive structure. The development of this guide has taken the approach of providing reasonable and prudent levels of design consistent with the state-of-the-art of engineering practice. The document is intended to be dynamic in nature; it is expected that it will be revised and updated by the experience gained through usage.

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# Appendix A

# Marine Concrete Repair

By

Douglas Burke

# Appendix X

**U.S. NAVY**
**SPECIFICATIONS AND GUIDELINES**
**For**

# MARINE CONCRETE REPAIR


**ABSTRACT**

The U.S. Navy is continually looking for the best methods and materials to repair its reinforced concrete waterfront structures. The goal is to identify methods and materials that provide at least 15 years of service. Corrosion of the steel reinforcement is the paramount failure mechanism for Navy waterfront facilities. Sometimes the repair area is so extensive that "patching" the concrete is not practical and replacement is the favored alternative. Therefore, this document addresses new construction criteria in addition to repair methods and materials. Corrosion activity manifests itself as cracks, spalling, delamination, and eventually the reduction of structural capacity and operational readiness. Corrosion mitigation methods and materials are emphasized through the use of low shrinkage cementitious repair materials, proper concrete cover, and proper surface preparation and placement techniques that result in durable repairs. Epoxy-coated rebar is recommended for new construction because it provides supplemental corrosion protection. Discussions, guidelines, specifications, and illustrative details are provided for inspection, new construction, and repairs above and below the waterline.



Prepared by:
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March 12, 1999

# CONTENTS

# PART A

# CONDITION ASSESSMENT AND INSPECTION

## SUMMARY

The average age of Navy waterfront structures is about 50 years and they typically exhibit deterioration due to rebar corrosion, cracking, and carbonation. Rebar corrosion is usually very active in the splash zone and at curbs, joints, and where concrete cover over the rebar is insufficient. Prior to repairs being made, a comprehensive inspection of the concrete is necessary to delineate the amount of concrete needing repair. Guidelines for condition assessment are contained in various industry standards.[1,2]

During the removal of the damaged concrete and while the contractor is "chasing" the corroded rebar to a point where it is not significantly corroded, it is typical to uncover areas that need to be repaired that were not identified in the initial inspection. Non-destructive inspection techniques and tools are not always adequate to predict the extent of repairs that need to be made. Because of the frequent occurrence of "going over budget" during repairs, it is common for planners and estimators to increase the estimated repair amounts when preparing the repair budget. Contingentency factors of two to four times are typically used by some planners. Even then, many repair projects go over budget.

## INSPECTION TOOLS

The degree of deterioration is often much more extensive than is first apparent. Many tools are available to collect data to make repair estimates, the usual tools are summarized below:

- Delaminations are usually detected by using a hammer or chain.

- Powder samples taken from the top and bottom deck are used to measure the degree of chloride contamination at the depth of the rebar. Values that exceed 1.5 pounds per cubic yard are at the threshold at which rebar corrosion may occur if sufficient moisture and oxygen are present.

- A pachometer (rebar locator) may be used to measure the depth of concrete cover over the rebar. Rebar that has less than 1.5 inches of cover is very likely to be corroded or will corrode.

[1] Military Handbook Maintenance of Waterfront Facilities Sept 1997 MO-104
[2] Guide for Making a Condition Survey of Concrete in Service ACI 201.1R-84

- A Schmidt Hammer may be used to approximate the concrete's compressive strength according to ASTM C 805 (Standard Test for Rebound Number of Hardened Concrete). Variations in surface strength may indicate areas of concrete that that are soft from carbonation or delamination.

- A petrographic analysis can be very useful to approximate the water-to-cement (w/c) ratio, quantify the cement paste-to-aggregate bond, and to identify other failure mechanisms such as alkali silica reaction and the formation of ettringnite. Higher w/c ratios are generally associated with greater permeability.

- A portable adhesion tester may be used to determine the concrete's surface tensile strength according to ASTM D 4541 (Standard Test Method for Pull-Off Strength of Coatings Using Portable Adhesion Testers). Concrete in good condition will exhibit tensile strengths between 300 psi to 500 psi. Surface condition is important information if coatings or membranes are to be applied as part of the repair. If coatings are used, a vapor emission test is necessary and the results are expressed in pounds of vapor per 1,000 square feet. Coating selection is dependent on vapor transmission.

# PART B

## NEW CONSTRUCTION
## (REPAIR BY REPLACEMENT)

Repairs to marine concrete facilities are typically required because of insufficient cover to the reinforcement or inadequate concrete quality. In other cases, the structure may have been damaged by ship impact or a seismic event. When repairs are very extensive, it may be more economical to replace all or part of the structure. Part B addresses the scenario of "repair by replacement."

# CHAPTER 1

# CONCRETE DURABILITY

# CONCRETE DURABILITY

## INTRODUCTION

This chapter has been extracted from an introduction to the subject of durability of marine concrete structures[1]. It addresses the general deteriorating mechanisms that may occur in concrete structures and fundamental guidelines for specifying durable materials for reinforced concrete in a marine environment.

All materials are vulnerable to destructive actions. The destruction can be slow or rapid, depending on the material and the surrounding environment. Materials that are very durable in one environment may deteriorate rapidly in another environment. The ability of materials to withstand destructive actions is expressed by the term durability.

The durability of one specific material naturally varies with the type of attack. Steel within concrete can be destroyed by electrochemical attack or corrosion, for which the concrete itself is completely unsusceptible. The concrete, however, can be affected by other destructive forces of chemical and physical origin, such as deterioration from different aggressive chemical substances, deterioration by means of frost action, deterioration by abrasion, and so on.

Concrete is a porous material, porosity being the presupposition for almost every form of deteriorating forces. Concrete may also be vulnerable to cracking due to thermal movements, shrinkage, and moisture movement. The pore system and small cracks make it possible for different substances to enter the interior of the reinforced concrete and attack the different elements that constitute the material. Water, oxygen, and chlorides can reach the steel reinforcement and destroy it by means of corrosion. Water that fills up the pore system and small cracks can freeze, expand, and deteriorate the concrete. The permeability of the concrete, therefore, is a main parameter governing the durability of concrete.

## STEEL REINFORCEMENT CORROSION

The most serious and probably most frequent type of deterioration of marine concrete structures in general is reinforcement corrosion. In a young concrete structure, the steel reinforcement does not corrode because of the alkaline nature of the surrounding concrete. The steel is said to be in a passive state. The adequacy of the alkaline protection is dependent upon the thickness of the concrete cover to the reinforcement, the quality of the concrete, the details of the geometry of the structure, the degree of chlorides in the concrete constituent materials, and external sources.[2 3] However, over a period of time the protecting environment may disintegrate and the passivity destroyed. This may happen by:

- Carbonation of the concrete cover caused by ingress of carbon dioxide

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[1] Draft report on "Durability of offshore concrete structures," prepared by Aker Maritime, Lars Bjerkeli, for the Office of Naval Research, Feb 1999.

[2] ACI 201.2R-92: Guide to durable concrete.

[3] ACI 222R-96: Corrosion of metals in concrete.

- Penetration of chloride ions to the reinforcement

Sound marine concrete has a very low permeability. A permeability coefficient of less than $10^{-12}$ m/s measured on drilled samples is required according to the Norwegian Petroleum Directorate[4]. Carbon dioxide penetration in concrete with this low permeability is so slow that carbonation is not an issue for marine concrete.

The required low permeability and the minimum concrete cover requirements according to Norwegian Standard NS 3473[5] are the main efforts implemented to attain a chloride ion level adjacent to the reinforcement that is low enough not to break the reinforcement passivity. Even if the steel passivity is broken, corrosion requires an electrolytic cell in which oxygen must penetrate the concrete cover and reach the steel. The low permeability of the concrete cover and the low diffusion rate of oxygen from the seawater significantly reduce the possibility of oxygen reaching the reinforcement.

If the concrete is cracked, a more rapid penetration of chlorides to the steel may take place. The effort to reduce this deteriorating effect to an acceptable level is limitation of the crack widths. The most stringent crack width limitations apply to the splash zone and the atmospheric zone. These zones are more or less continuously subjected to wetting and drying and there is enough oxygen to sustain the corrosion process. In addition, frost action may occasionally occur.

A more relaxed crack width limitation applies to the permanently submerged zone. In this zone, cracks that permit water ingress tend to close themselves by additional hydration of the cementing materials, autogenous healing, and deposition of filling materials. Leaching of calcium hydroxide and other soluble substances from the cement paste fill up the cracks and may close them completely. Deposition of materials such as argonite and brucite resulting from reactions of the seawater and the cement will also seal cracks.

An extensive introduction to the influence of crack widths and self-healing of cracks on durability may be found in Jakobsen, et al.[6]

Deterioration of concrete due to corrosion may result in significant cracking because the products of corrosion (rust) occupy a greater volume than the steel and exert substantial stresses on the surrounding concrete. The outward manifestations of rusting include staining, cracking, and spalling of the concrete.

## CORROSION PROTECTIVE MEASURES

The concrete mix design, sufficient cover to the reinforcement, and limitation of crack widths have already been mentioned as important factors to prevent reinforcement corrosion. These factors are introduced as requirements in the design basis.

Experience from pouring of concrete in full-scale production proves that a theoretically durable concrete mix design, tested and qualified by laboratory tests, may be difficult to apply in "real life" construction. This may be because of the applied production technique and equipment or due to high reinforcement densities, weather conditions, and or some other reason.

[4] Norwegian Petroleum Directorate: Regulations for loadbearing structures in the petroleum activities. 1992

[5] Norwegian Standard NS 3473. "Concrete Structures, Design Rules", 4th edition 1992.

[6] S.Jakobsen, J,Marchand and B.Gerard: Concrete cracks I: Durability and self healing - A review. Proceedings of the second international conference on concrete under severe conditions, CONSEC –98, Tromsø, Norway

To avoid these potential problems, trial mixes should be qualified under full-scale production situations. Efforts to obtain the most optimal curing conditions must also be tested out prior to being applied in production. Non-structural cracks, i.e., cracks that are a result of unfavorable curing and hardening conditions rather than tensile stresses in the concrete caused by external loads, are often a larger concern than structural cracks. According to Mehta,[7] thermal shrinkage and drying shrinkage is the primary cause of cracks in many new concrete structures. These cracks are related to high early-strength concrete mixes that typically have a relatively high content of fine ground cement. In addition, the cement often has a relatively high sulphate and alkali content. These types of cracks are not controlled by the amount of minimum reinforcement.

Preventing the chlorides from entering the concrete is an additional effort that may be applied to reduce the potential corrosion problem. An extensive list of methods for controlling and monitoring corrosion may be found in ACI 22R-96.[8]

## FREEZE THAW CONSIDERATIONS

The mechanism for freezing and thawing marine concrete is the same as for onshore concrete. It consists of two parts, one related to the material ("the strength") and one to the environment ("the load").

The material-related part is concerned with how much water within the concrete is sufficient to cause frost damage. This material property is called the critical degree of water saturation $S_{cr}$.

The environment-related part is concerned with how much water is present within the concrete. This part is called the actual degree of water saturation $S_{act}$. For frost damage to occur, $S_{act}$ has to be as high as $S_{cr}$. or higher.

The following recommendations apply to concrete that will be exposed to a combination of moisture and cyclic freezing.[9]

- Design of the structure to minimize exposure to moisture
- Low water–cement ratio.
- Appropriate air entrainment
- Quality materials
- Adequate curing before first freezing cycle
- Special attention to construction practices

## SULPHATE ATTACK

Sulphate ions in the seawater can attack the calcium hydroxide in the cement paste, the final reaction product being ettringite. As ettringite is very voluminous, the result can be a volume increase with a subsequent disruption or softening of the concrete.[8] The concentration of

[7] P.K.Mehta: Durability – Critical issues for the future. Concrete International, July 1997
[8] ACI 222R-96: Corrosion of metals in concrete.
[9] ACI 201.2R-92: Guide to durable concrete.

sulphate ion in seawater can be increased to high levels by capillary action and evaporation under extreme climatic conditions.

Low permeability, low water–cement ratio (w/c < 0.4), and a low $C_3A$ content in the cement are the key factors to obtain acceptable sulphate resistance of concrete.

## ALKALI-SILICA

Reactive siliceous minerals in the aggregates can be attacked by alkaline hydroxides in the pore water, the hydroxides derived mainly from the oxides of sodium and potassium in the cement paste. The reaction product is an alkali-silicate gel. The gel can absorb water resulting in an increase of volume and internal pressure, which eventually may cause cracking of the concrete. At low temperature, the reactions may become dormant.

The expansion of the gel is dangerous only for certain combinations of reactive aggregates, alkaline hydroxides, and water in the concrete. If the content of alkaline hydroxides is low (the equivalent amount of sodium dioxide should not be higher than 0.6% according to ASTM), the reaction is small and of no significance. Also, if the content of reactive silica aggregates is low or high, or the particle size is very small or very big, the reaction is small and of no significance.

In general, the reactivity of the aggregate should be tested before being recommended for use. Outline of test methods, criteria for judging reactivity, and recommended procedures to be used with alkali-reactive aggregates may be found in footnote 8.

## ALKALI-CARBONATE

Some dolomitic limestone aggregates can react with the alkaline hydroxides in the pore water. The result is an expansion similar to that occurring as a result of the alkali-silica reaction described above. The expansion causes a network of pattern or map cracks usually most strongly developed in areas of the structure where the concrete has a constantly renewable supply of moisture. The expansion can be very severe, and has led to extensive problems onshore in Canada and the U.S. Fortunately, reactive carbonate rocks are not very widespread and can usually be avoided.

An outline of test methods, criteria for judging reactivity, and recommended procedures to be used with alkali-reactive aggregates may be found in ACI 201.2R-92.[8]

## SULPHUR REACTION

Sometimes aggregates contain sulphur compounds, usually as sulphates and sulphides, which can be the cause for deleterious expansions. Sulphides, together with water and oxygen, can produce reaction products similar to those which can be produced by sulphates. Careful testing and examination of the aggregates will usually indicate the presence of such reactive impurities and their use in concrete avoided.

## LEACHING

Under special circumstances, submerged concrete may undergo some leaching of the more soluble substances (mainly calcium hydroxide) from the cement paste. The actual leaching is very small and probably takes place mainly in cracks, where it serves the useful purpose of helping to close the cracks.

## WATER TIGHTNESS

The water permeability of uncracked concrete is governed mainly by the cement paste and the contact zone between the aggregates and the paste.

Concrete with a water/cement ratio below 0.45, which is the case for the North Sea marine structures, is for all practical purposes non-permeable with respect to transport of liquids through the section thickness. Tightness is therefore primarily a matter of avoiding through cracks that may arise during hardening of the concrete or during platform construction and operation. A minimum requirement for amount of ordinary reinforcement is equally important for the efficient distribution into many small cracks rather than a relatively few larger cracks. In addition, special attention has to be paid to construction joints.

## CONSTRUCTION JOINTS

Potentially, construction joints can act as waterways through a concrete structure. To avoid this completely, it is necessary to have skilled labor and well-proven routines to treat the joints properly.

The routines vary quite a lot, depending on the situation. Typically, the concrete surfaces at all joints are thoroughly cleaned prior to placing adjoining concrete. Then, if possible, a layer or concrete with increased cement content and workability is placed against the joint, followed immediately by ordinary concrete, with vibration through the first layer.

If water tightness of a construction joint is required, procedures must be prepared to test the joint for leakage before construction work is completed. Observed leakage is repaired with epoxy injection. Precautions can be taken by pre-installation of epoxy injection tubes in the casting joint and injection of epoxy after hardening of the concrete.

## CLIMATIC EFFECTS

The temperature will affect the rate of the deteriorating chemical processes involved, but the processes themselves will be basically the same. Other important factors that may influence the deteriorating processes are the oxygen content and the salinity of the seawater. Different experiences in various climatic zones are often related to quality control during construction, the local material sources applied, and the conditions during construction; not to the environment itself.

In Fookes, et al.[10] the climatic situation world-wide is divided into four types: (1) hot wet, (2) hot dry, (3) temperate, and (4) cool (which also includes freezing). Their characterization is shown in Figure 1, together with a world ocean salinity and temperature chart.

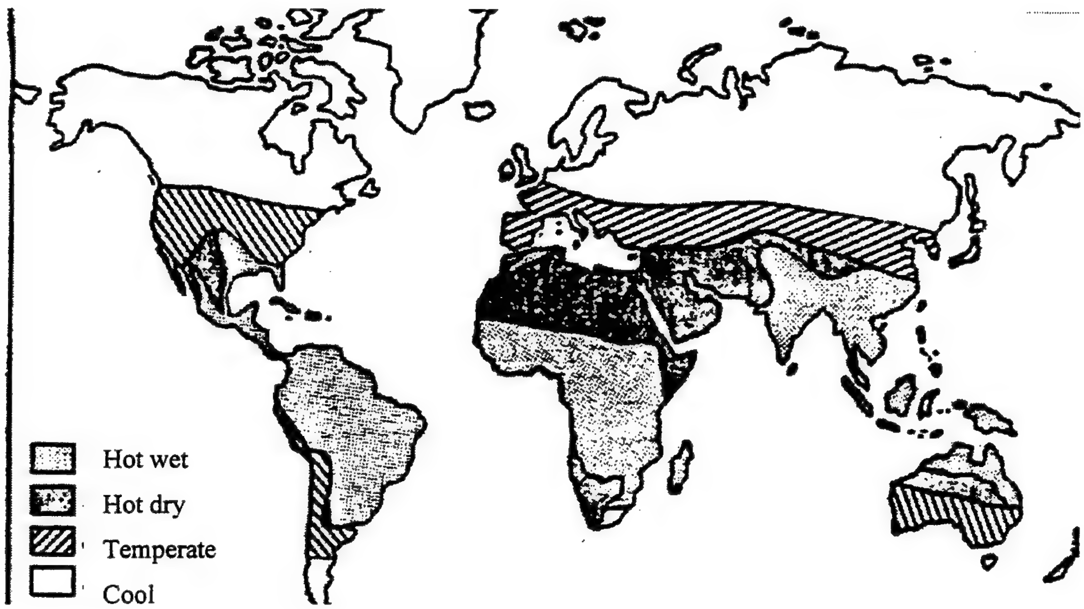


Figure 1. World climatic regions and world ocean and salinity and temperature chart [9]

## HOT-WET REGIONS

According to ACI committee 305[11] the definition of hot weather is any combination of the following conditions: (a) high ambient temperature, (b) high concrete temperature, (c) low relative humidity, (d) high wind velocity, and (e) solar radiation. A comprehensive presentation of recommended practice and precautions for hot weather concreting may be found in RILEM TC 94CHC[12] and footnote 10.

## SERVICE LIFE CALCULATIONS FOR MARINE CONCRETE STRUCTURES

Analytical models have been developed to be able to calculate the service life of marine concrete structures. The basis for the models is that reinforcement corrosion due to chloride ingress is the main deteriorating mechanism. The traditional assumption has been that chloride ingress into concrete obeys Fick's second law of diffusion for a semi-finite medium with constant exposure, and that there is a critical value of the chloride content in the concrete, $C = C_{cr}$, leading

[10] P.G.Fookes, J.D.Simm and J.H.Barr: Marine concrete performance in different climatic environments, Marine concrete '86 London Sept.1986.

[11] ACI Committee 305; Hot weather concreting. ACI chapter 305 R-91 American Concrete Institute

[12] RILEM TC 94CHC (1993): Concrete in hot weather environments. Draft "Part I: Influence of the environment on reinforced concrete durability. Part II: design approach for durability".

to the corrosion of the steel. Typical parameters in such models are chloride background content, chloride content on the exposed surface, exposure time, penetration depth, and diffusion coefficient. Recent research has shown that the diffusion resistance improves over time and that this has to be taken into account in analytical models, i.e., the diffusion coefficient is time dependent.

## SUMMARY

The key requirements for new marine concrete structures are low initial cost, low maintenance costs, and a long service life. The main requirements for production of durable marine concrete in all environments are summarized as follows:

- Make concrete as dense as possible
- Use fairly rich mixes, i.e., 350 kg to 450 kg of cement per $m^3$ of concrete.
- Use appropriate cover to the reinforcement.
- Use sulphate-resisting cement and non-reactive aggregates.
- Keep chloride content at a minimum in the concrete mix
- Consider the use of air-entraining agent to improve frost resistance if required
- Avoid thin sections and complicated geometry for properly pouring of the concrete.
- Apply the highest standards of workmanship and supervision
- Consider the effects of ambient temperature and environment during preparation of procedures for casting and curing.

# CHAPTER 2

# MIX DESIGN

# MIX DESIGN

## INTRODUCTION

This chapter is an extract[1] that describes some very significant developments in concrete technology and site practices have taken place over the past decades enabling improved concrete qualities to be produced on site. By means of selected constituent materials and mix proportions, high strength or high performance concrete can be produced to uniform and predictable levels of quality which will ensure long maintenance free service life under rough and hostile conditions.

Concrete mix design is an interactive process where selected constituent materials are combined to meet the technical requirements of the design and, at the same time, meet the practical performance requirements of the chosen plant and construction procedures on site. The design requirements apply primarily to the hardened concrete (strength, durability, ductility, density), whereas the site performance requirements apply to the fresh concrete (workability, pumpability, curing).

The process starts in the laboratory or with existing records with a view to identify suitable constituent materials. The requirements are laid down in relevant national and international Codes or Standards, supplemented with additional project-specific requirements. Tests may also be required to evaluate the mutual compatibility of individual materials.

The concrete strength and other mechanical properties are a function of the w/c, or water to binder, ratio. A low w/c ratio is required both for strength and durability reasons. To achieve this, while maintaining adequate workability and a moderate cement content, is often the key to a successful mix design. (The term w/c ratio is used throughout this report to define the ratio between water and total cementitious material, i.e., cement + silica fume + PFA or other pozzolans.)

Heat of hydration, or the temperatures generated during curing and hardening, sets up thermal stresses and may lead to undesirable cracking of the young concrete and impaired durability. The type and dosage of cement is the key factor in this respect and a low cement content consistent with the required strength and w/c ratio is desirable. In this respect, a combined binder of Portland cement, PFA, and/or silica fume may be an attractive option. Special curing measures may also be required to limit temperatures and temperature gradients in the early phase.

The target mean strength of the concrete required to achieve the stipulated characteristic value depends on the uniformity of production (coefficient of variation) which can be maintained. The efficiency of the Quality Assurance/Quality Control (QA/QC) system therefore also affects the mix design process.

The above demonstrates the loop linking material properties to site procedures. The choice of mix proportions is therefore an interactive process where site trials constitute an essential element. The experience from similar projects and from field experience with the proposed constituents provides valuable input and the recommendations contained in the present report should be seen in this light.

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[1] Draft report on "Concrete mix design," by Aker Maritime, Tom Moksnes, for the Office of Naval Research, Feb 1999.

It should also be emphasized that the chosen mix proportions must be sufficiently robust to stand up to changing site conditions. High performance concrete is generally sensitive to fairly minor changes in material properties and procedures. A 'hands on' approach to concrete production with immediate response and adjustments to the mix as the need arises imposes requirements on the efficiency of the quality control system and the competence of the operators.

In the past there was a tendency to focus on the strength properties at the expense of equally important properties such as durability, workability, and weight.

## DURABILITY

Concrete in seawater has adequate and satisfactory durability provided attention is paid to a few simple rules of mix design and construction. The essential requirements are the use of high quality concrete with low permeability and high frost resistance, and adequate and uniform concrete cover to the reinforcement.

Corrosion of the reinforcement is the most fundamental concern in concrete sea structures (rather than concrete deterioration), and ensuring that the steel is surrounded by adequate thickness of low permeability concrete will prevent corrosion. In simple terms, corrosion will not occur if neither oxygen nor seawater carrying chlorides can penetrate to the steel and cause a lowering of the pH and the creation of an electrochemical circuit.

- **The quality and minimum thickness of the concrete cover is the major durability factor.**

Cracks caused by shrinkage, creep, thermal gradients, and loads may also render the steel vulnerable to seawater ingress and subsequent corrosion and must be dealt with during design and construction. These factors are affected by the concrete composition as well as by the site procedures related to placing and curing. Adequate water spray or membrane curing should always be applied to the fresh concrete.

Other durability concerns for concrete in sea structures may include alkali-aggregate reactivity (AAR), sea water reactivity (sulphate and chloride reactions with the cement compounds), and frost resistance in the case of concrete exposed to freezing and thawing. The total chloride content in the fresh concrete needs to be restricted and the mixing water should always be fresh and potable.

For marine structures and particularly for unknown aggregate sources, petrographic examination by thin section microscopy should always be performed

Concrete deterioration due to sea water is the result of separate reactions between sulphates and chlorides present in the sea water and the cement compounds tricalcium aluminate $C_3A$ and portlandite $Ca(OH)_2$. A well-compacted high performance concrete with a low w/c ratio will provide adequate durability in the marine environment. Cement with a low $C_3A$ content is recommended for concrete exposed to aggressive types of sulphate attack. Some caution should be exercised in less aggressive environments due to the effect this may have on other and equally important concrete properties. A moderate $C_3A$ content of 5 – 7 % is commonly specified for concrete sea structures and has worked well.

Freeze-thaw resistance is dealt with by air entrainment, using air entraining admixtures to achieve the desired size and spacing of the air voids. It should be kept in mind that air

entrainment reduces the compressive strength and that stable air voids are not always easily achieved in highly workable (superplasticized) concrete.

## CONSTRUCTABILITY

Pumping is an expedient and cost efficient method of placing concrete in large and tall structures and pumpability of the concrete becomes a key factor in the mix design. Also, tall structures are often built by slipform construction that imposes special requirements on the fresh concrete. Finally, high strength concrete is commonly used for structures that are densely reinforced and prestressed and therefore demand highly workable mixes. This adds up to the fact that the properties of the fresh concrete are essential considerations in the mix design process. The main quality parameters for fresh concrete, commonly labeled constructability, are:

- **Workability (flow characteristics)**
- **Stability (lack of segregation and water separation)**
- **Open time (pot life)**

Essentially, what is required is a high slump (superplasticized concrete), a cohesive mix with little or no bleeding and segregation during pumping, compaction, and setting, and a mix with the ability to retain its fresh characteristics until the mix is placed and compacted. The setting time, i.e., the time after which the concrete can not be vibrated or remoulded and starts to harden, is critical for the rate of slipforming and is significantly affected by the choice and dosage of admixtures. A summary of the main quality challenges and the chosen remedies is shown in Figure 2-1.

| | |
|---|---|
| **HIGH STRESSES**<br>30m design waves.<br>High hydrostatic pressures.<br>Dynamic loads (earthquakes).<br>Severe loading in some construction phases, e.g., extreme water pressure on the cell walls during submergence for mating. | **REQUIREMENTS:**<br>High performance constituent materials of uniform and predictable quality.<br>Characteristic 28 day cube strength up to 80 MPa.<br>Acceptable fatigue properties |
| **DEMANDING CONSTRUCTION**<br>Dense reinforcement (300 kg/m$^3$ +).<br>Large volumes, tight tolerances.<br>Pumping to heights over 200m.<br>Large scale slipform construction. | High slump (250 mm)<br>No bleeding or segregation.<br>Acceptable and verified pumpability.<br>Adjustable and predictable setting time. |
| **HOSTILE ENVIRONMENT**<br>Sea water<br>Steel corrosion.<br>Concrete deterioration.<br>Freeze thaw damage. | Low w/(c+s) ratio (0.35 – 0.40).<br>Cement (binder) content min.400 kg/m$^3$ in splash zone.<br>Permeability coeff. < $10^{-12}$ m/sec.<br>Water intrusion (ISO/DIS 7031)< 25mm (target <15mm)<br>Curing temp. max. 65-70°C.<br>Concrete cover min 50mm to ord.<br>Reinforcement, min 70mm to prestressing steel.<br>Air entrainment in freeze/thaw conditions. |

Figure 2-1. The main concrete quality requirements adopted in the North Sea [2]

[2] A.K.Haug, M.Sandvik. Mix design and strength data for concrete platforms in the North Sea. 2. International Conference on Performance of Concrete in Marine Environment, St.Andrews, Canada, August 1988.

## CEMENT

Different cements will have different water demands for a given cement paste consistency and will therefore affect the rheological properties of the concrete mix. The $C_3A$ content of the cement affects several properties and the suggested value of 5.5 is a compromise between its effect on sulphate resistance, chloride initiated rebar corrosion, heat of hydration, and early loss of slump.

- **A major step in the mix design process is to identify a suitable cement or cement/pozzolan combination for the prevailing high performance concrete requirements.**

## AGGREGATES

Aggregates constitute 70% of the volume of the concrete mix. For low or medium strength concrete a wide range of natural or crushed aggregates will have adequate properties provided they comply with the mandatory requirements. This is not the case for modern high strength/high performance concrete where the strength of the aggregate particles will affect the ultimate strength and deformation characteristics of the chosen mix. Similarly, the particle size, grading, and mineral composition will significantly affect the water content required for a given workability, and hence the w/c ratio and durability.

The properties of the aggregates depend on their geological origin, their geological history in terms of transportation and sedimentation, and their quarrying, processing, and handling methods.

This development of tailoring the sand grading, including the content of silt and fines, has had a profound effect on the properties of the fresh concrete, the w/c ratio, and the strength properties. These properties are also affected by the particle size, shape, and texture of the coarse aggregate which can also be modified by selected processing methods. The mechanical properties of the aggregates play an important role in controlling the strength, E-modulus, ductility, and fracture mechanics properties of high strength concrete. Variations in the E-modulus of the coarse aggregate may significantly affect the E-modulus of the concrete (by as much as 25 to 50%). For equal uniaxial compressive strength levels, different rock types may cause 15 to 40% differences in tensile and flexural strength.

The durability aspect concerning aggregates is linked to their chemical stability and to their secondary role in determining the concrete density through their effect on w/c ratio and workability. A main concern regarding chemical stability is the ability to identify potentially alkali reactive aggregates and thin section microscopy should be conducted on any unknown source of aggregates.

## LIGHTWEIGHT AGGREGATES

Different types of LWA concrete have been developed and been found to enhance properties such as durability and energy absorption. LWA concrete comes in a wide range of

densities and strengths and there are no clear lines of division separating LWA concrete from normal density (ND) concrete.

Some lightweight aggregates with lower water absorption may be suitable for pumping provided the aggregates are thoroughly soaked in water prior to mixing. One such material is Stalite, a proprietary material of the rotary kiln expanded shale type produced in the U.S. Stalite was used on the Hibernia GBS project in Newfoundland for the production of large quantities of 80 MPa MND concrete.[3]

The high porosity and water absorption of the LWA aggregates is a major factor in mix design and production. The water absorbed during mixing must be uniform and predictable to achieve consistent properties.

LWA concrete has lower thermal conductivity and lower heat storage capacity than ND concrete and the temperature rise due to heat of hydration will be higher. Cooling measures may need to be implemented in large sections.

All the design properties of high strength LWA concrete have been studied and many differ to some extent from those of ND concrete. A summary of the general trend that has been observed is given below in Figure 2-2:

| Advantages of LWA Concrete | Disadvantages of LWA Concrete |
|---|---|
| Reduced weight and improved buoyancy.<br>Better crack behaviour from shrinkage, creep and thermal expansion.<br>Reduced cracking from deformation loads.<br>Better energy absorption from impact loads.<br>Lower permeability.<br>Improved durability and corrosion resistance.<br>Improved freeze-thaw resistance.<br>Equal or better fatigue behaviour. | Reduced resistance to locally concentrated loads and need for confining reinforcement.<br>Lower E-modulus, brittle failure mode.<br>Higher cement content for given strength.<br>Higher heat of hydration.<br>Liable to spalling of cover under HC fire.<br>Need to control water content and absorption for consistent workability.<br>More demanding to batch, place and cure.<br>Higher cost per $m^3$. |

Figure 2-2. Observed relative merits of high strength LWA compared to ND concrete.

It should be emphasised that the test results and observations reported above need to be verified for the chosen concrete mix design and lightweight aggregate. LWA concrete is well suited for marine applications provided all the different properties, such as the increased brittleness, are accounted for in the detailed design of the structure.

## ADMIXTURES

Chemical admixtures, and particularly the water reducing types, are essential to the production of high strength/high workability/low w/c ratio concrete. Major improvements in their performance have been achieved in recent years.

[3] C.G.Hoff, R.Walum, J.K.Weng, R.A.Nunez: The Use of Structural Lightweight Aggregates in Offshore Concrete Platforms, Proceedings International Symposium on Structural Lightweight Aggregate Concrete, Sandefjord Norway June 1995, Norwegian Concrete Association, Oslo.

Today, a range of very efficient proprietary high resolution water reducing admixtures (HRWRA) are available and are used in dosages of 1 - 2% of the cement weight. Site trial mixes need to be performed to decide the best product(s) and dosage for the specific purpose as the total mix composition and the batching process and procedure may affect the results.

If air entrainment is deemed necessary to achieve frost resistance in freeze/thaw conditions, air-entraining admixtures can be used to obtain the desired volume of very small micropores. Acceptable frost resistance is commonly evaluated by measurement of the air void system or by freeze/thaw test procedures. The need for freeze/thaw protection of high strength concrete in the North Sea is an ongoing discussion and research suggests that non air entrained low w/c ratio concrete has a high resistance to freeze/thaw deterioration.

Air entrainment will lead to a loss of compressive strength, and this must be taken into account when air entrainment is contemplated and decided. Air entraining admixtures are commonly used in small dosages of 0.1% of the cement weight and work better with some HRWR admixtures than with others.

Set retarding admixtures are used when the rate of construction is such that the set retarding effect of the HRWR admixtures is not sufficient. Chloride free set accelerators have also been developed for a quicker set but have so far only had limited application for the large offshore projects.

Common to all admixtures is that their performance is dependent upon a number of mix design and site specific factors and that they are marketed under proprietary brand names. Site trials are therefore essential prior to selecting the most efficient brands and dosages.

## SILICA FUME

Modern high strength concrete benefits by a small dosage of silica fume and is considered essential for high strength MND and LWA concrete.

Silica fume (microsilica) is a by-product of the ferro-silicon industry. It is an extremely fine powder with grain size less than 0.1 μm and specific surface of about 20,000 $m^2$/kg. It is a reactive pozzolan, consisting of 90% amorphous $SiO_2$ and acts as a very effective filler. Its fineness and its reactivity account for the beneficial effects of silica fume in high performance concrete.

Silica fume in moderate dosages will increase the compressive strength, increase the resistance to corrosion of the embedded steel, and improve the resistance to segregation of a high slump concrete. Dosages of 5 – 8% are considered very beneficial for all high performance LWA concrete. Silica fume can be blended into the cement at the mill or distributed and added as a slurry during batching. There are side effects, particularly at high dosages, related to stickiness and plastic cracking if proper curing measures are not implemented.

## FIBER REINFORCEMENT

Fibers may be used to increase the tensile strength and the toughness of concrete, an example of the improved ductility that can be achieved by the addition of 1% fiber. Fibers will also increase the impact strength and reduce shrinkage. The fibers may be steel, carbon, glass, polymers, and other synthetic materials. The length, shape, and mechanical properties of the fibers affect the way they impact on the properties of the concrete. Fiber contents of 1 – 5% by

volume have been used for special applications such as sprayed concrete, precast units, special hard wearing pavements and caps for driven piles.

The inclusion of fibers significantly affects the properties of fresh and hardened concrete and introduces special requirements on the batching plant and procedures. Fibers have not been used for the mass concrete on any of the large offshore projects described in this report. Although they may appear to offer advantages as 'crack reinforcement', their impact on constructability and on construction procedures is negative. The addition of even small quantities of fiber reinforcement should be considered very carefully and thoroughly tested on site before they are adopted for improved ductility and durability of large marine concrete structures.

## COATINGS

Polymer impregnation of concrete may improve the resistance to freezing and thawing, the abrasion resistance, and the general durability in an aggressive environment. The application is difficult and demanding and would not be practical to perform on very large structures. Polymer coatings have not been applied to the offshore projects described in this report. Polymer modified mortar coatings were, however, applied in some critical areas on the Northumberland Strait Bridge Project.[4]

Epoxy coatings have been applied to some of the North Sea structures to provide additional protection in the splash zone. The epoxy was applied by trowel to the freshly slipformed surfaces of the shafts and subsequent inspections have revealed that the coatings have been successful.[5]

## THE SIGNIFICANCE OF THE W/C RATIO

The general relationship between the w/c ratio and compressive strength is good for the 0.35 to 0.40 range of w/c ratio commonly specified for offshore concrete structures. Above a w/c ratio of 0.40 the volume of capillary water and air pores increases rapidly and contributes to a more porous cement paste and reduced durability.

Permeability tests show that the coefficient of permeability of cement paste increases exponentially as the w/c ratio increases above 0.50.[6] The low permeability associated with low w/c ratios gives added durability which in some cases is a more significant feature of high performance concrete than the strength itself.

The low w/c ratio benefits both strength and durability and the range of 0.35 to 0.40 is commonly adopted for high performance concrete. A w/c ratio below 0.35 requires high binder contents and very high dosages of HRWR admixtures to achieve a satisfactory workability and may have undesirable side effects with respect to early cracking. Such concrete has also been

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[4] E.W.Tromposch, L.Dunaszegi, O.E.Gjørv, W.S.Langley: Northumberland Strait bridge project-strategy for corrosion protection. Proceedings 2. International Conference on Concrete under Severe Conditions, CONSEC 98 Tromsø, E&FN Spon, London.

[5] R.Aarstein, O.E.Rindarøy, O.Liodden: Effect of Coatings on Chloride Penetration into Offshore Concrete Structures. Proceedings, CONSEC 98, Tromsø. E&FN Spon, London.

[6] A.M.Neville, J.J.Brooks: Concrete Technology, Chapter 14, Longman, London 1987.

found to be vulnerable to inadequate site procedures and to require more skilled and experienced operators.

## SUMMARY

No mix design can be completed without site experience and full scale site trials. The chosen mix proportions for high performance concrete need to be continuously monitored and adjusted as the work progresses to account for variations in the local conditions. From a virtually total focus on *high strength* working close to the level of feasibility, industry is shifting their focus to *high performance.*

It should be remembered that high strength concrete is less tolerant to variations in site conditions and practices than ordinary concrete and the quality obtained depends to a larger extent on the knowledge and skill of the operators.

The designer has the freedom to choose within a broad range of options to achieve the desired properties of the fresh and hardened concrete while still operating within the realm of high strength/high performance concrete. The fundamental requirements for durability are high quality constituent materials, a low water/binder ratio, good constructability facilitating placing and compaction, proper curing, and adequate cover to the steel.

The term "high quality constituent materials" requires special attention and implies thorough durability testing according to the relevant Codes as well as mechanical testing where compliance with the Code is only the first step in the process. Full scale site tests on concrete mixes and mock-up tests on site procedures will be needed for a finalization of the concrete mix design. Once the mix constituents and proportions have been selected, a quality assurance system needs to be established and implemented that ensures continuous monitoring and adjustments to the mix to suit the prevailing and changing conditions on site. An important aspect of this system is the interface with the site procedures for formwork and rebars to ensure that the concrete can be properly placed and compacted and that the specified concrete cover can be maintained.

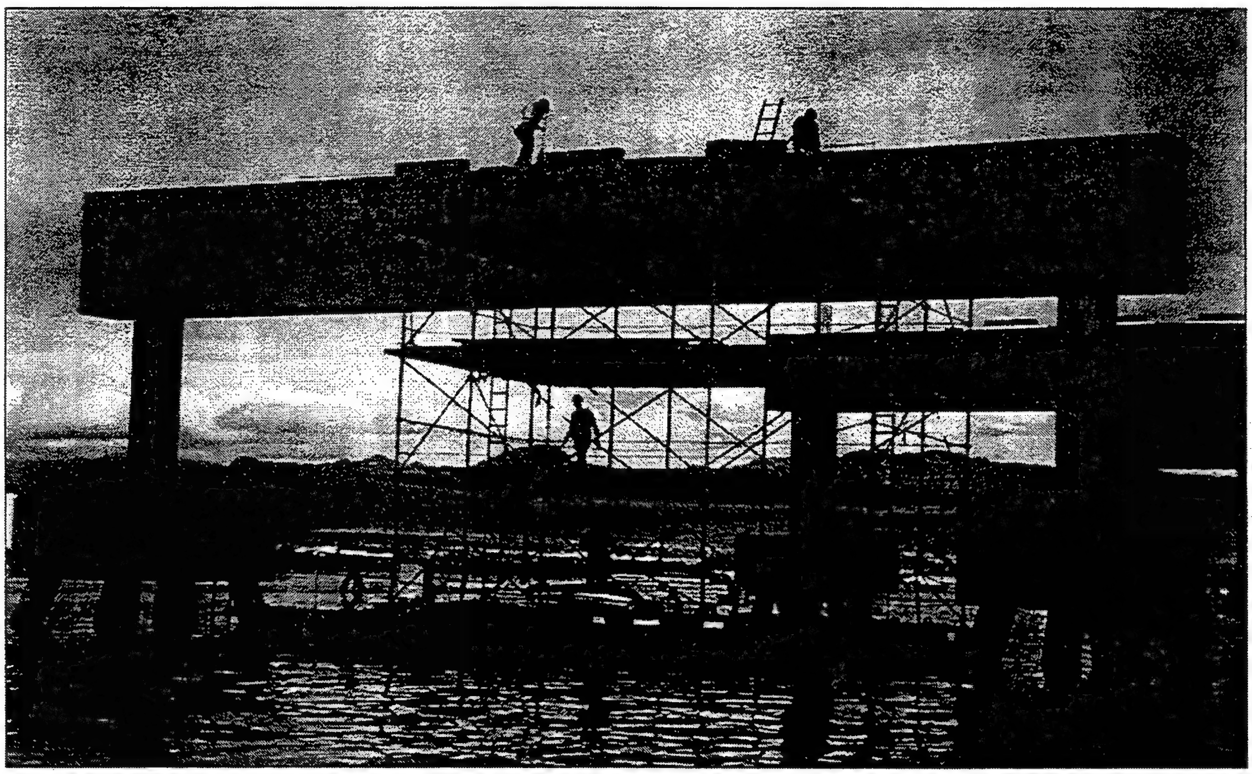
# CHAPTER 3

# USE OF NEW GENERATION EPOXY-COATED REBAR IN THE ADMIRAL CLAREY BRIDGE

## USE OF NEW GENERATION EPOXY-COATED REBAR IN THE ADMIRAL CLAREY BRIDGE

### INTRODUCTION

The design and construction of the Admiral Clarey Bridge exemplifies the use of durable reinforced concrete in a marine environment. Planners, designers, and builders must pay great attention to the many critical factors that ultimately contribute to the durability of the reinforced concrete. This paper provides a brief summary of some of the important concrete material issues related to performance with particular emphasis on supplemental corrosion protection using new standards for prefabricated epoxy-coated steel rebar.



**Construction of the Admiral Clarey Bridge, Pearl Harbor, Hawaii**

### THE ADMIRAL CLAREY BRIDGE

The Admiral Clarey Bridge connecting Ford Island to the Pearl Harbor Hawaii Naval Complex was dedicated April 15, 1998. The 4,700-foot long bridge is one of six reinforced concrete floating bridges in the world. The 650-foot moveable span is the longest in the world.

The request for proposals (RFP) for the design/build contract was developed with the assistance of a number of people and organizations. For the bridge concept, the RFP relied heavily on studies by the Pacific Division Naval Facilities Engineering Command's (PACNAVFACENGCOM) Planning Department. These included various Ford Island Access studies accomplished in 1987/88 and the Final Environmental Impact Study in 1990.

The RFP provided specific design criteria, which were developed mostly by PACNAVFACENGCOM's Design Division engineers with assistance from the following:

- Naval Facilities Engineering Command, John Headland, Coastal Engineering.

- Washington State Department of Transportation, Myint Lwin, floating pontoon section and concrete.

- Federal Highway Administration, Raymond McCormick, highway/bridge.

- Naval Facilities Engineering Service Center (NFESC), Douglas Burke, prefabricated fusion-bonded pipeline-type epoxy-coated reinforcement.

Because rebar corrosion occurs much faster in a tropical environment, such as Hawaii, it was particularly important that emphasis be placed on the design of the concrete materials to provide long term durability. To maximize concrete durability, these design decisions were made:

- The use of 5 percent silica fume was recommended by Mr. Lwin based on his experience and success in using silica fume on the most recently constructed Washington State floating bridges.

- The use of a maximum allowable water-to-cement ratio (w/c) of 0.38 was based on waterfront engineering practices using locally available Hawaiian concrete materials.

- The use of a zero tension under service load criterion was based on the State of Hawaii Department of Transportation requirement for all bridges in Hawaii.

- The use of prefabricated fusion-bonded epoxy-coated rebar was a difficult decision to specify since the Navy's new standard was still under development by NFESC and the increased cost was uncertain. Ultimately, 4,600,000 pounds of epoxy-coated mild reinforcing steel was used to construct the bridge.

## COST/BENEFIT

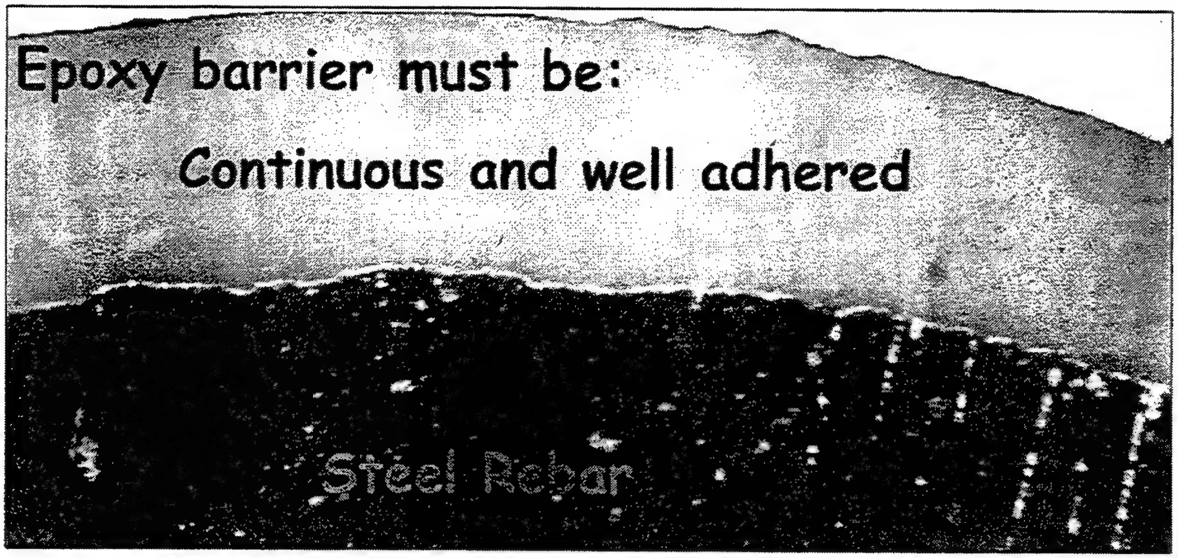
For the Admiral Clarey Bridge a cost comparison of using plain steel rebar versus prefabricated epoxy-coated rebar was done. The predicted costs were $1.20/pound for plain rebar compared to $1.60/pound for coated rebar, installed. Therefore the additional cost amounted to ($0.40) x (4,600,000) = $1,840,000. Since the cost of the total project was $86 million, the premium to use this technology was 2.1 percent. Use of high quality concrete materials and workmanship with proper concrete cover should provide a 50-year service life. Field performance evaluations and accelerated laboratory tests of coated rebar indicate that the technology will provide a substantial increase in life performance. Rebar life extension on the order of 20 to 40 years is a rational expectation.

## EPOXY-COATED REBAR DEVELOPMENT

The decision to use epoxy-coated rebar in new Navy construction was based on extensive evaluations that began in 1984, when the Office of Navy Research tasked the Naval Civil Engineering Laboratory to conduct long-term field evaluations. Test specimens were suspended in a marine intertidal zone for 76 months at Key West, Florida to rank the relative performance of popular corrosion control methods. Damage-free epoxy-coated rebar performed best. Results from this study were presented by the Concrete Reinforcing Steel Institute in their Research Series 2 report of July 1994.

Despite the good performance in the Navy's long-term field tests, the Florida Department of Transportation and other agencies had found moderate to severe corrosion much earlier than expected on some marine structures using epoxy-coated rebar. By 1994, much controversy surrounded the use and performance of epoxy-coated steel reinforcing bars produced and placed in accordance with current specifications. Consequently, the Navy Criteria Office funded the NFESC to identify the failure mechanisms in current practices and to develop a new standard in cooperation with industry experts. This effort resulted in an Interim User's Guide for Prefabricated Epoxy-Coated Rebar for Oceans and Other Severe Environments (PROSE). The document included two new Navy Facilities Guide Specifications (NFGS), 03201 and 03202, and recommendations for a quality control program. The Navy Criteria Office identified candidate construction projects to incorporate the new generation of epoxy-coated rebar. Two Navy submarine piers were constructed, one in Pearl Harbor, Hawaii, and the other in New London, Connecticut. NFESC monitored the construction of each project and evaluated the cost and constructablity. Both projects proved highly successful and the differential costs were about 2 percent higher for each with respect to the overall construction cost. The toughness of the new epoxy powder formulation developed by 3M proved exceptionally good, requiring very few repairs after shipping, storage, and placement. The bridge also included small sections of epoxy-coated rebar coated with epoxy powder formulated by Akzo Nobel and Herbert's-O'Brien, which appeared to be equally durable.

The American Society of Testing and Materials (ASTM) used the Navy's draft specifications as a basis for the development of ASTM A 934/A 934M published in July 1995, "Standard Specification for Epoxy-Coated Prefabricated Steel Reinforcing Bars." Mr. D. Burke of NFESC is the current chairman of the ASTM Subcommittee A01.05 task group for development and revision of coated reinforcement standards. In February 1998, the NAVFAC Criteria Office published, for the first time, a definitive guide for Marine Concrete, NFGS 03311. Included is a requirement to use prefabricated epoxy-coated reinforcing steel according to the new ASTM Standard.




**Magnified View of Epoxy-Coated Rebar**

## IMPORTANT FEATURES FOR ENHANCED PERFORMANCE

There are many important features of the new technology for prefabricated epoxy-coated rebar contained in the ASTM Standard that contribute to improved performance. Some of these are:

- All of the rebar is prefabricated to final size and shape prior to coating. This avoids stress cracks in the coating and loss of coating adhesion in the bend areas during post fabrication, which has been a typical site for corrosion.

- Since the coating no longer needs to be flexible, new epoxy powder formulations can be used. These formulations are more durable and resistant to the intrusion of corrosive elements.

- Extensive quality control tests must be performed on every batch of coated rebar, including cathodic disbondment tests for coating adhesion. This requirement greatly reduces problems with underfilm corrosion.

- All visible defects in the coating must be repaired prior to concrete placement. This minimizes the number of locations in the barrier coating that might otherwise become corrosion sites.

In addition to the recommendations contained in the ASTM standard, NFESC strongly recommends that:

- Coated rebar is not mixed with plain rebar in the structure. This avoids the possibility of creating a large corrosion cell if there is electrical continuity between the coated and uncoated steel.
- Coated rebar should not be used in structures that are subject to large impact loads and in areas where the steel is severely congested (e.g., 50 percent or more of the cross section is steel). Because of the lack of adhesion of the cement paste to the epoxy coating, the concrete that covers the reinforcement may disbond when subject to impact loads, which was reported when a reinforced concrete component was accidentally dropped.
- Designers should not specify the use of coated rebar that exceeds number 11 (2-3/4" diameter) until definitive data is available that addresses the effect on bond and anchorage.

## CORROSION ACTIVITY

The purpose of providing supplemental corrosion protection, such as an epoxy coating, is to reduce the rate of rebar corrosion, thus increasing the time before corrosion related repairs are necessary. This is accomplished in two important ways:

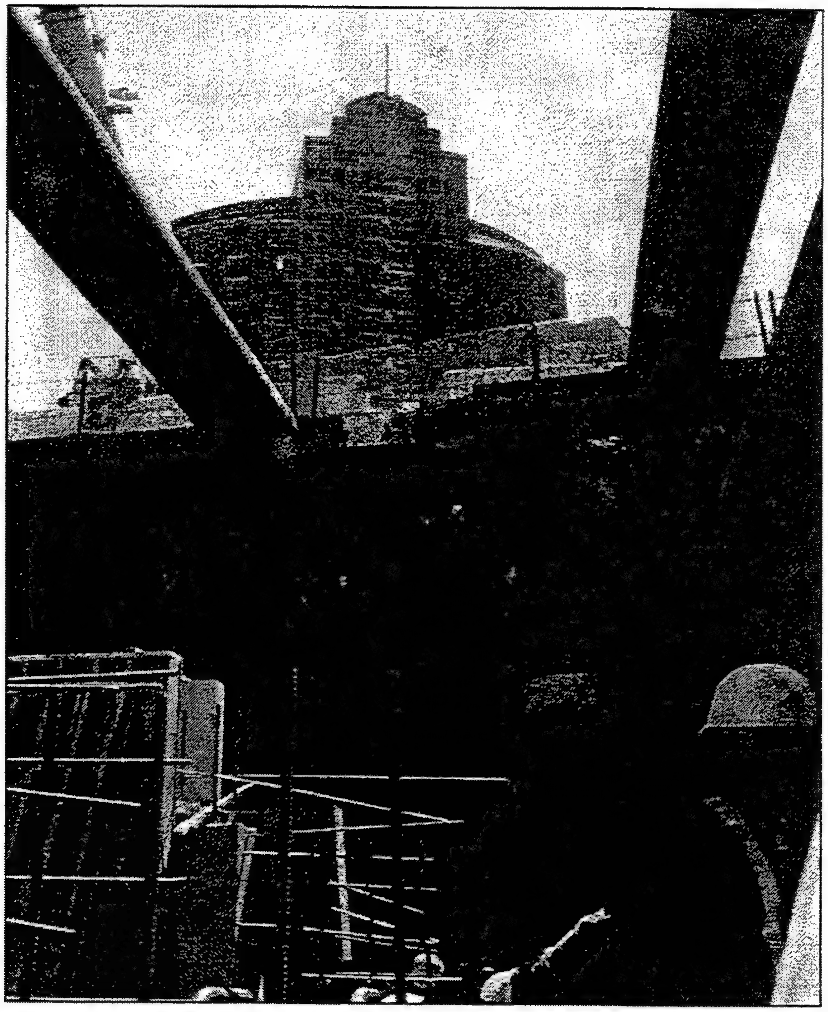
- If the quality of the concrete is compromised in any manner that results in cracks, increased concrete permeability, or reduced concrete cover, then chloride, oxygen, and water will find their way to the rebar sooner than expected. An excellent barrier coating on the steel will extend the time before corrosion will take place.
- Eventually corrosive elements will reach the rebar regardless of the concrete materials used and the quality of the workmanship. When the chloride contamination reaches the threshold level necessary for the initiation of steel corrosion, the presence of a highly impermeable well-adhered barrier coating with a minimum number of defects will retard the potential for corrosion activity in the steel reinforcement.

## CONCLUSION

Concrete durability in a marine environment requires strict attention to many important aspects of planning, materials, design, and workmanship. Life performance of marine structures can be enhanced by the use of prefabricated epoxy-coated steel reinforcing bars with good coating adhesion and no visible damage to the coating. Construction of the Admiral Clarey Bridge exemplifies the use of these design and construction principles.

## OTHER PROJECTS USING EPOXY-COATED REBAR

Several projects within the Navy and in the private sector have used the new standards for prefabricated epoxy-coated rebar, such as, the Muni-Metro Turn Back in San Francisco, California and the Long Beach Aquarium in Long Beach, California. In June 1997, the technology was reviewed and adopted by the California Department of Transportation (CALTRANS) for reinforced concrete structures in contact with sea and brackish water.



**Construction of Muni-Metro Turn Back, San Francisco, California**

## ADDITIONAL INFORMATION

Additional information about the design and construction of the Admiral Clarey Bridge is contained in an excellent and comprehensive article by Michael Abrahams and Gary Wilson featured in the PCI Journal July/August 1998 issue. For more information about the use of prefabricated epoxy-coated reinforcement, please contact Douglas Burke at 805-982-1055 or burkedf@nfesc.navy.mil.

# CHAPTER 4

# CALCIUM NITRITE ADMIXTURE

## CLACIUM NITRITE ADMIXTURE

### BACKGROUND

Due to the severity of a marine environment and the likelihood for corrosion of the steel reinforcement, designers must specify high quality concrete and an adequate concrete cover over the rebar. In addition, the use of a corrosion protection system can provide for additional corrosion protection. Epoxy-coated steel rebar, galvanized steel rebar, and calcium nitrite admixture all provide beneficial effects. ("Performance of Epoxy-Coated Rebar, Galvanized Rebar, and Plain Rebar with Calcium Nitrite in a Marine Environment," D. Burke, July 1994.)

### PASSIVATION OF STEEL IN CONCRETE

Due to the alkaline environment of concrete, a protective passive layer of iron oxide forms on the surface of the steel rebar. This passive layer is composed of ferrous ($Fe^{2+}$) and ferric ($Fe^{3+}$) oxides (refer to Figure 1). Ferrous oxides are susceptible to chloride attack, whereas, ferric oxide resists chloride attack. It is important to note that the ingress of carbon dioxide, water, and oxygen can also contribute to the breakdown of the protective passivation layer. This is called carbonation corrosion.

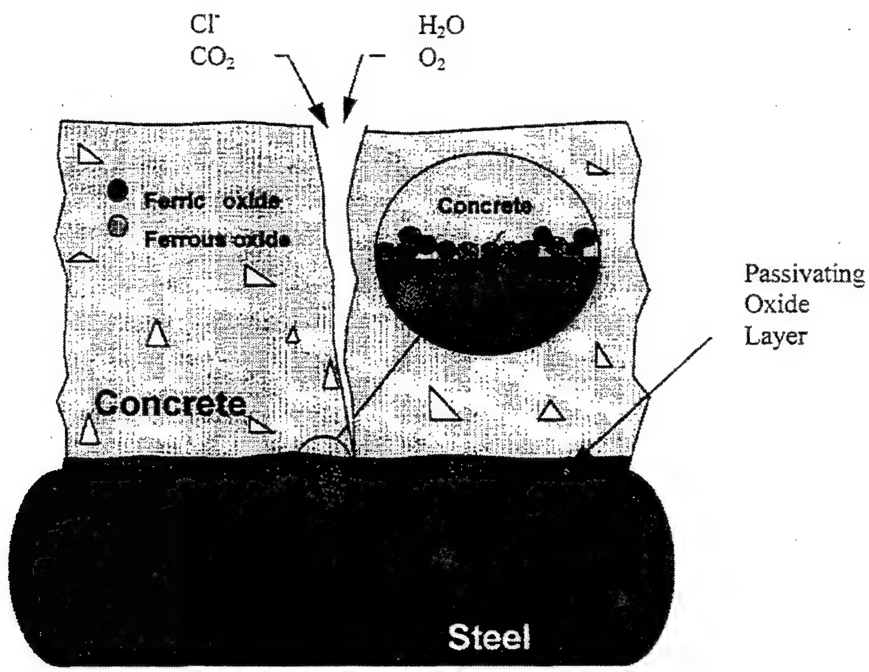



Figure 1. Passivation of steel in concrete.

## CHLORIDES AND CORROSION

Corrosion of steel reinforcement in concrete occurs in locations where chloride intrusion has weakened and/or destroyed the passive layer. This commonly occurs at the ferrous oxide sites.

The chloride level necessary to initiate corrosion of steel in concrete is approximately 1.5 $lb/yd^3$. The time it takes to reach this level of contamination at the depth of the rebar is dependent on the exposure, quality of concrete, depth of concrete cover over the reinforcement, and the number and depth of cracks that develop in the concrete. Due to the high availability of chloride ions in a marine environment, the chloride corrosion threshold can be reached within a few years.

## CALCIUM NITRITE ADMIXTURE

To protect the rebar and passive layer from attack, calcium nitrite can be introduced into the concrete mix during batching as an admixture. It is introduced into the concrete at either the precast or cast-in-place concrete operation. Calcium nitrite reduces the steel rebar's susceptibility to corrosion by increasing the oxide surface concentration, hence strengthening the passive layer. The duration of protection offered by calcium nitrite is dependent on the dosage used and corrosion rate.

The amount of calcium nitrite the specifier selects is a function of concrete quality and the environment to which the structure is to be exposed. Therefore, these factors need to be considered when selecting a dosage of calcium nitrite. General guidelines as to the dosage amount for certain environments are provided by the manufacturer. A marine environment typically has a recommended calcium nitrite (30% solution) dosage of 4 to 6 $gal/yd^3$. The most common dosage is 4.5 $gal/yd^3$ of concrete. American Concrete Institute has published two applicable references: ACI 212.R-91 Chemical Admixtures for Concrete and ACI 222-89 Corrosion of Metals in Concrete.

Calcium nitrite has not detrimental effects to hardened concrete properties. Both neutral set and accelerated set versions are available to accommodate project requirements.

## CAST-IN-PLACE CONCRETE

Cast-in-place concrete is frequently used in the construction of seawalls, piers, and berthing docks. Cast-in-place concrete quality is dependent on many factors. Some of these factors are water-to-cement ratio (w/c), total cement content, aggregate type and gradation, curing conditions, and job site weather conditions. A corrosion protection system is required for direct marine exposure. When the use of more than one protection system is specified for the same structural component, this is referred to as a redundant system. The use of a redundant corrosion protection system properly addresses the durability required for these marine-exposed structures. Calcium nitrite and epoxy-coated rebar is an example. The calcium nitrite helps protect the steel where defects occur in the coating. One may wish to use a redundant system to increase confidence in long term durability.

## PRECAST CONCRETE

Precast concrete is commonly used in marine piles and substructure deck members. Due to the controlled manufacturing conditions, precast members are typically of high quality and uncracked. In use, these concrete members are exposed to direct sea water and therefore will benefit from supplemental corrosion protection. The use of epoxy-coated rebar and a calcium nitrite admixture should be considered.

## CONCLUSIONS

Research has shown that the addition of calcium nitrite admixture to concrete can slow the onset of corrosion for both cracked and uncracked test specimens. Calcium nitrite admixture has been used commercially since 1978. The dosage of the product needs to be determined for each project based on the predicted chloride diffusion rate and desired design life. To be successful with calcium nitrite, be sure to follow the manufacturer's recommendations and instructions for application. All marine concrete should use quality concrete with a low water-to-cement ratio concrete (<0.40), proper concrete covers, and proper curing to maximize performance. These recommendations are consistent with ACI concrete practice guidelines, which should be followed.

## POINT OF CONTACT

If you would like further information on this subject, contact Mr. Douglas Burke, Naval Facilities Engineering Service Center, Code ESC63, at (805) 982-1055 or DSN 551-1055, or e-mail at burkedf@nfesc.navy.mil.

# CHAPTER 5

# NFGS-03311 – MARINE CONCRETE

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DEPARTMENT OF THE NAVY
NAVAL FACILITIES
ENGINEERING COMMAND
GUIDE SPECIFICATION

NFGS-03311
15 February 1998

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# NFGS-03311

# MARINE CONCRETE

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Preparing Activity: NFESC EC DET (00CE4)

| | Typed Name & Reg. | Signature | Date |
|---|---|---|---|
| Prepared by: | M.S. Chawla, P.E. | | |
| Approved by: | P.N. Bolton, P.E. | | |
| Approved for NAVFAC: | Carl E. Kersten, R.A. | | |

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AMSC N/A

AREA FACR

**Note: This document is available through John Lynch, Criteria Office, Code 15, EFD, LANTDIV. Phone: 757-322-4207, DSN 262-4207; e-mail Lynchjt@efdlant.navfac.navy.mil.**

# CHAPTER 6

# ABS GUIDELINES FOR MOB

# ABS CLASSIFICATION GUIDE
# CONCRETE MATERIALS FOR DoD MOBILE OFFSHORE BASE (MOB)

Prepared by:
Naval Facilities Engineering Service Center
Douglas F. Burke
805-092-1055 email: burkedf@nfesc.navy.mil

This draft revised on: 17 January 1999

Table of Contents

10.3.5 Concrete Materials for Aircraft Traffic Surfaces



10.3.5 Quality Control:

## 10.3 Reinforced Concrete for MOB

The reinforced concrete used in MOB construction shall be of sufficient durability and strength to meet the intended service conditions and performance life as expressed in the MOB Mission Requirment Statement. A satisfactory methodology for design, material selection, placement, and quality control shall be developed. Consideration shall be given to the use of prestressed lightweight concrete and modified density concrete. Development of the criteria and specification shall reflect international performance records, codes, standards and research data.

### 10.3.1 General Requirements

#### 10.3.1.1 Scope

The scope of this section is to set forth requirements for prestressed lightweight concrete and modified density concrete to construct the MOB. These requirements are also applicable for a "hybrid" MOB, constructed with structural steel and prestressed concrete. Emphasis is placed on the use of structural modified density concrete and a methodology for material selection that will result in a durable concrete structure. The term "concrete MOB" implies a concrete hull using columns and pontoons that support a steel deck. On top of the steel deck, the aircraft-wearing surface may also be constructed with concrete. Design and construction of the MOB will present unique challenges; issues related to design and construction are addressed in other documents. [1,2]

#### 10.3.1.2 Exposure Conditions

The MOB will operate in a severely aggressive environment and will be continuously exposed to ocean currents, wave action, seawater spray, tidal action and submerged conditions. Concrete exposure for MOB may be determined from the exposure classes defined in appropriate standards.[3,4,5,6] Materials criteria and specifications shall be developed to produce durable concrete to resist the failure mechanisms summarized in section 10.3.1.4 Design Methodology.

#### 10.3.1.3 Durability Requirements

The use of prestressed concrete and reinforced concrete has been found appropriate for construction of ships and offshore structures in severe marine environments. [7,8,9] The United

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[1] U.S. Navy Mobile Offshore Base ARCOMS Concept Study

[2] Structural Analysis and Design, 7 April 1998, Document No. 9019-ANC-JD-RN-0005

[3] Building Code Requirements for Structural Concrete, ACI 318, Chapter 4. American Concrete Institute POB 9094 Farmington Hills. MI 48333.

[4] British Standards Institution (1997a). pr EN206. Concrete - performance, production and uniformity. Draft for Public Comment, BSI Document 97/104685, Committee Reference B/517.

[5] Norwegian Standard NS 3473 E

[6] Hobbs, D.W., Minimum requirements for durable concrete, British Cement Association 1998

[7] ACI, State-of-the-Art Report on Barge-Like Concrete Structures Reported by ACI Committee 357, ACI 357.2R-88

States Navy commissioned the construction of at least 15 concrete ships during World War I including the USS Selma, USS Atlantis and USS Polias. During World War II more than 100 concrete ships were constructed. Several of these ships have been the subject of material durability investigations that have concluded that concrete can provide long term durability in a marine environment. [10,11] However, "many structures built in accordance with codes and guidelines of recommended practice have shown deterioration much before their intended service life".[12] Therefore, it is mandatory to incorporate a high level of quality in design, material selection and construction. To accomplish the necessary durability it is recommended that the designer take a holistic approach to durability.[13] In addition, the development of the criteria and specifications should incorporate recommendations contained in the following documents.

- ACI 357 Concrete for Offshore Structures
- ACI 357.2R Concrete for Barge-like structures
- Canadian Standards S-374 Structures for Offshore and Frontier Areas, Concrete
- ACI committee reports on structural lightweight aggregate concrete[14,15]
- ACI 318 Building Code[16]
- U.S. Navy specifications for marine concrete[17]
- European Standard pr EN 2061 (draft)
- Norwegian Standard NS 3473 E.
- CEB Bulletin 238 - New Approach to Durability Design

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[8] Severin, L. et. al. "Troll A Gas Production Platform – Implications of 50-70 Years Life Span, OTC 8412, Offshore Technology Conference, 1997

[9] Anderson, A. R., " A 65,000-Ton Prestressed Concrete Floating Facility For Offshore Storage Of LPG"

[10] Heun, Raymond C., "Concrete Ships -- Long Forgotten," Concrete International, April 1995, pp. 54-56.

[11] Bremmer, Theodore W.; Holm, Thomas A.; and Morgan, Dudley R., "Concrete Ships -- Lessons Learned," Performance of Concrete in Marine Environment, AP-163, American Concrete Institute, 1996, pp. 151-169

[12] Mehta, P. K. "Point of View, Durability – Critical Issues for the Future". Concrete International, July 1997, pp. 27-33.

[13] Mehta, P. K. and Gerwick, B.C., "Concrete in the Service of Modern World." Proceedings of the International Conference on Concrete in the Service of Mankind, University of Dundee, Scotland, June 1996.

[14] ACI Committee Report 304 "Batching, Mixing, and Job Control of Lightweight Concrete", 1991.

[15] ACI Committee Report 213B "State-of-the-art Report on Structural Lightweight Aggregate Concrete for Bridges and Other Exposed Structures" Draft October 1996.

[16] ACI Committee Report 201.2R "Guide to Durable Concrete," Manual of Concrete Practice, American Concrete Institute, Farmington Hills, MI, 1997 pp. 33-37.

[17] Naval Facilities Engineering Service Center, Naval Facilities Guide Specification NFGS-03311, February 1998

- FIP Manual of Lightweight Aggregate Concrete

10.3.1.4 Failure Mechanisms

A long service life with no major repairs in a marine environment demands the use of high quality reinforced concrete. The design methodology used shall account for the failure mechanisms that may manifest as deterioration of the concrete. Typically, the most likely deterioration mechanism for marine exposure is corrosion of the steel reinforcement. The concrete must be resistant to deterioration from the following mechanisms over the performance life of the structure.

- Concrete shall be of high tensile strain capacity to be able to resist the formation of cracks due to the volume changes, and shall be reinforced and/or prestressed to provide adequate toughness to resist the crack propagation due to the various mechanical, physical and environmental loadings.
- Concrete shall be resistant to chemical disintegration caused by alkali-aggregate reactions, sulfate attack, and delayed ettringite formation.
- Concrete shall have adequate resistance to freezing and thawing attack.
- Resistant to wear and to the formation of cracks and crack propagation from impact loads, chains, cables, minor collisions, wave action, vibration, fatigue, abrasion, freeze and thawing, and other physical and environmental loading.
- Resistant to the ingress of environmental agents such as air, water, chlorides, sulfides and carbonates.

10.3.1.5 Basis of Design

"One important limitation of conventional concrete, even of good quality, is the presence of microcracks, capillaries and micro-capillaries into which water is able to penetrate; sucked in by surface tension forces or driven by an external hydrostatic pressure."[18] Intrusion of water is a significant factor affecting the rate at which many concrete failure mechanisms progress. The use of high performance lightweight concrete or modified density concrete can provide superior impermeably to structure subject to hydrostatic pressure when compared to normal weight concrete..[19] A durable concrete MOB must be highly resistant to the ingress of moisture. Chloride ions also migrate through permeable concrete by diffusivity, even without actual flow of water. Therefore, the design and materials selection must strive to minimize the ingress and migration of water, oxygen, chlorides, sulfides, and carbon dioxide into the concrete. In time, these substances are directly responsible for the deterioration of the concrete and corrosion of the reinforcing steel. Because the penetration of these substances is inevitable, proper concrete cover over the reinforcing is critical to achieve long life.

[18] Roy, Salil, K. and Northwood, Derek, O. "Admixtures to Reduce the Permeability of Concrete" SP 170-13 p. 269

[19] Dr. Lar, PhD thesis (incomplete footnote at this time)

Conventional reinforced concrete is designed to crack in tension; consequently, the reinforcement that coincides with these cracks may be exposed to corrosive agents soon after the structure is put into service. Alternatively, structures built with prestressed concrete will have no cracks transverse to the direction to prestress under service conditions. Temporary crack widths during construction should be less than 0.30mm provided the cracks are crossed by reinforcing steel and will be relieved from stress in service.

Prestressed concrete requires supplemental steel reinforcement to control the formation and propagation of numerous crack mechanisms. This is provided by the use of plain steel reinforcement for stirrups, spiral coils, bolsters, T-headed bars, anchor dowels, stitch bolts and transverse reinforcement.

Lightweight concrete bridges and other exposed structures have a proven history of long-term performance.[20] Compared to structures using normal weight aggregates, LWC will have greater resistance to microcracking because of their lower modulus of elasticity, lower coefficient of thermal expansion/contraction and strain compatibility at the aggregate-cement matrix interface.[21] Additionally, prestressed lightweight concrete structures are capable of providing good energy absorption.

The designer of MOB shall use the following as a minimum basis for design to provide durable performance for the design life of the MOB.

- In the Serviceability State, concrete that is fully submerged should have crack widths less than 0.25mm. Membrane shear cracks and transverse cracks in the hull shall be less than 0.15mm in the splash zone. Through cracks in external bulkheads and ballasted tank walls should be less than 0.10mm.[22]
- The hull shall be prestressed longitudinal and transversely (if necessary) so as to prevent the development of repeated cyclic tension under the waves in the serviceability limit state.
- Pontoon and other portions of the hull should be sloped and scuppers should be adequate to prevent ponding of water.
- Minimum reinforcement in both directions, and on both faces should be provided in order that a crack, which opens for any reason, will be restrained by rebar below yield stress. The potential tension zone is defined as ($c + 7\varnothing$) where $c$ = cover thickness and $\varnothing$ = diameter of rebar transverse to the potential crack. The area of required reinforcing is $A_S = A_C f_{Ct} / f_y$, where $A_C$ is the area of concrete in a unit length of thickness equal to the potential tension zone, $f_{Ct}$ is the flexural tensile strength of the concrete at age 7 days, and $f_y$ is the yield

[20] ACI Committee Report 213B "State-of-the-art Report on Structural Lightweight Aggregate Concrete for Bridges and Other Exposed structures" page 1, Draft October 1996.

[21] Vaysburd, A.M. 1992. Durability of Lightweight Concrete and its Connections with the Composition of Concrete, Design, and Construction Methods. Proceedings of the International Symposium on Performance of Lightweight Concrete, ACI SP-136 pp. 295-318

[22] Gerwick, Ben, C. "Construction of Prestressed Concrete Structures" Second Edition, Wiley Professional Series, 1993, p. 141

strength of the reinforcement. For example, for concrete with $f'_c$ of 60MPa at 28 days and $f_y$ = 400 MPa, this results in a requirement of 0.8% reinforcement in the tension zone.

- Adequate prestressing and/or reinforcing steel shall be provided to effectively resist in-plane (membrane) shear. This shall take into account the reduction in efficiency of orthogonal reinforcement for resisting diagonal cracking.
- Fatigue of prestressed concrete under cyclic loading of waves can be effectively resisted by keeping the stresses in the serviceability Limit State below $0.5f'_c$ compression and zero tension. Fatigue must be considered since it has been well established both by tests and experience in the North Sea that fatigue for concrete members, especially those submerged in water, under repeated waves is a serious problem. [23].
- Through-thickness reinforcement shall be provided in all elements subjected to high compressive stresses in the extreme Limit State in order to provide confinement. Such through thickness reinforcement is also required in the sides of hulls to resist transverse (out-of-plane) shear and impact.
- Adequate prestressing and/or reinforcing steel shall be provided to effectively resist in-plane (membrane) shear. This shall take into account the reduction in efficiency of orthogonal reinforcement for resisting diagonal cracking.

#### 10.3.1.6 Materials for Construction

MOB design life can be accomplished by using lightweight aggregate and modified density concrete, and implementing sound construction practices and quality control.[24,25,26,27,28] The following considerations for the design, criteria development and specifications are provided.

[23] The Lacy V. Murrow floating bridge in Lake Washington developed cracks over the years that led (or contributed) to its sinking. Now the Evergreen Point Bridge, also in Lake Washington, has started to develop similar cracks at the same age. It is currently undergoing a major retrofit.

[24] Neville, A., "Point of View: Is Strength Enough? Maintenance and Durability of Structures" Concrete International, November 1997 pp.52-56

[25] Lamond, J. F., "Designing for Durability" Concrete International, November 1997 pp. 34-36

[26] Mehta, P. K. "Point of View, Durability – Critical Issues for the Future". Concrete International, July 1997, pp. 27-33.

[27] Gerwick, Ben, C. "Construction of Prestressed Concrete Structures" Second Edition, Wiley Professional Series, 1993 pp. 137-177

[28] The Japan Society of Civil Engineers, Guidelines for Durability Design of Concrete Structures (Draft), 1996

- Use prestressing and closely spaced reinforcement to prevent and or minimize crack width.
- Use lightweight aggregates that are resistant to water absorption, limit absorption to 8%.
- Use controlled processing of the aggregates.[29]
- Use proper aggregate grading.
- Limit size of the coarse aggregate to 20 mm.
- Ensure that the concrete has low water permeability.
- Use good design and construction practices to minimize thermal gradients during curing.
- Restrict maximum temperature of the concrete during curing to 65°C.
- Use rounded or chamfered corners to minimize impact damage.
- Use 6 to 10% of aluminate in the cement.
- Use a $SO_3$ cement content of less than 0.4% by mass.
- Use fly ash, ground blast furnace slag, and microsilica.
- Use small diameter reinforcing steel.
- Use chemical admixtures and low w/cm ratio to enhance the air-void system, placement, compaction and workability of the concrete mix.
- Use adequate quality concrete cover over prestressing steel, 75mm minimum. Cover over conventional reinforcement shall be 50-mm minimum.
- At least 50% of the concrete coarse aggregate shall be lightweight. Fine aggregates may be normal weight. Aggregates must be sound and resistant to the mechanisms of deterioration.
- The lightweight concrete and the prestressed tendons must remain sufficiently dimensionally stable.
- Provide for proper and complete consolidation of the fresh concrete.
- Use proper finishing and curing methods.
- Seal all joints.
- Consider concrete surface protective coatings.
- Use qualified and comprehensive inspection to ensure that the construction conforms to the specifications and the intent for a durable structure.
- All concrete which will be exposed above water in normal service shall be properly air entrained to prevent freeze-thaw attack.

#### 10.3.1.7 Reinforced Concrete Design Criteria

The design criteria shall be developed from these ABS Guidelines and applicable world-wide industry standards. Note that durability is determined by many factors, especially impermeability, and is not directly related to concrete strength.[30] Draft design criteria based on performance characteristics are presented in section 10.3.2.2.

[29] Sandvik, M. et. al. "Chloride Permeability of High-Strength Concrete Platforms in the North Sea" CANMET/ACI International Conference on Durability of Concrete, Nice, France, May 1994

[30] Gjorv, O.E. "Steel Corrosion in Concrete Structures Exposed to Norwegian Marine Environment" Concrete International, April 1994 pp. 35-39

### 10.3.1.8 Material Specifications

The specifications are the link between the vision of the Department of Defense and the construction of the project. They provide a written vehicle between the Navy and Contractor to meet the Navy's needs. To accomplish this objective, the specifications must be easy to understand and to implement, therefore good specifications are fundamental to the project's success. The designer must write the specifications for the project. The specifications will be performance based in their nature.

## 10.3.2 Criteria for Reinforced Concrete

### 10.3.2.1 Final Criteria

The final criteria shall be developed using a systems approach. Various tools are available to aid in the prediction of the concrete durability. Examples include trial mixes, durability studies (computer models and laboratory testing) and finite element analysis for predicting thermal stresses. Extreme caution must be exercised in accepting claims and test data provided by material suppliers. A full-scale mock up of critical sections should be made in order to verify the analytical result. It is important to observe and measure any cracking due to thermal strains, shrinkage strains, and prestressing.

### 10.3.2.2 Draft Criteria

Table 10.1 presents <u>draft</u> design criteria for MOB. Industry standards contained in the draft criteria include American Concrete Institute (ACI), Norwegian Standard (NS), American Standards for Testing and Materials (ASTM). CEB, New Zealand Concrete Structures Standard (NZS).

Table 10.1 Draft Design Criteria for MOB

| Property | Criteria | Relevant Specifications |
|---|---|---|
| Concrete Durability | w/(c+m) splash zone < 0.35<br>w/(c+m) atm. & submerged < 0.37<br>Min. (c+m) (splash zone) 400 kg/m$^3$ (675 lb./cu yd)<br>Min. cement content (atmospheric and submerged) 350 kg/m$^3$ (590 lb./cu yd) | ACI 318, ACI 605, ACI 306<br>NS 3473 E<br>CEB Bulletin 238<br>pr EN206<br>BSI Document 97/104685<br>NZS 3101:1995 |
| Ec | Ec: 20-25 Gpa (2900-3600 psi) LWA. | ASTM C469 |
| Constructibility | Minimum bleeding and segregation<br>Consistent quality and constituents<br>Control of batching and distribution<br>Slump 200-250 mm with plasticizers | ACI 211.3<br>ACI 143 |
| Cement | Blended cement | ASTM C 150, ASTM C 595, ASTM C 845.<br>Japanese Belite-Rich Type A<br>HS 65 Norcem |
| Mixing Water | Chloride ions < 650 ppm<br>Sulfate ions < 1000 ppm<br>No oil, No nitrates | |
| Aggregates | Lightweight Aggregates<br>Normal weight Fine Aggregates<br>Resistance to abrasion and degradation<br>Resistance to disintegration by sulfates<br>Particle shape and surface texture<br>Grading<br>Bulk unit weight<br>Absorption and surface moisture<br>Aggregate constituents<br>Resistance to alkali-aggregate reactivity | ASTM C 330<br>ASTM C 33<br>ASTM C 131, C 535, C 779<br>ASTM C 88<br>ASTM C 295, D 3398<br>ASTM C 117, C 136<br>ASTM C 29<br>ASTM C 70, C 127, C 128, C 566<br>ASTM C 40, C 87, C 117, C 123, C 142, C 295<br>ASTM C 227, C 289, C 295, C 342, C 586 |
| Chemical Admixtures | High range and normal plasticizers as required to obtain workability and consolidation. | ASTM C 494<br>ASTM C 260 |

| | | |
|---|---|---|
| | Entrained air content 3-7% | |
| Mineral Admixtures | Silica fume 6-8% by mass of cement<br>Fly ash Type C, F or N (optimum dosage to be established by experimental program). | ASTM C 1240<br>ASTM C 618<br>ASTM C 989 |
| Compressive Strength | 55-70 MPa (7900-10,000 psi) lightweight aggregate concrete (LWA) | ASTM C 469 |
| Tensile Strength | Determined by the structural design. | ASTM C496 (splitting)<br>ASTM C78 (flexural) |
| Thermal Stresses | Maximum concrete curing temperature 60°C<br>Maximum gradient 20°C per 300 mm (36°F per 12 in.) | ACI Committee 207 |
| Chloride Contamination | Not to exceed 0.9 kilograms per cubic meter at 75mm depth after 40 years (1.5 pounds per cubic yard ) | Laboratory tests using same materials proposed for construction and Fick's Second Law of Diffusion |
| Conventional Curing | Continuous moist curing for 7 days | ACI 308<br>ACI 305R |
| Steam Curing | 60°C maximum | |
| Control Surface Cracks width | During construction, 0.30mm (if crack is crossed by rebar)<br>Serviceability state: 0.15mm in splash and atmospheric zones, 0.20mm submerged. | ACI 318<br>Norwegian Standard NS 3473 E.<br>CEB Bulletin 238 - New Approach to Durability Design |
| Fatigue Resistance | Unless justified by detailed analysis, place limits under serviceability limit state of 0.5 f'c compression , zero tension | |
| Freeze/Thaw Resistance | *Note: Conventional freeze thaw tests and methods are inadequate. Proposed criteria must be carefully prepared and evaluated with regard to research by the Canadian Dept. of Minerals and US Waterways Experiment tests at Treat Island, Marine* | |

| | | |
|---|---|---|
| Resistance to Abrasion | For lightweight aggregate, use of Silica Fume is essential. | ASTM C 779 |
| Depth of Carbonation | Not to exceed 1mm per year | Indicator test pH < 10<br>ASTM STP 169A |
| Resistance to Impact | *Under evaluation* | |
| Prestressing steel | 7-wire strands of cold drawn wire, stress-relieved, low relaxation. 75mm concrete cover minimum.<br>Ducts. Corrugated plastic ducts, not less than 0.25mm thickness, and anchorage caps, Grout for tendons. To be neat cement grout, with up to 15% fly ash allowed, and including a non-bleed thixotrophic admixture. | NS 3420, chapter L45 |
| Supplemental steel rebar | Grade 400 to grade 500 Mpa weldable reinforcing steel.<br>Fusion Bonded Epoxy Coated Steel Reinforcement for all stirrups, confinement, bolsters, T-headed anchors, and cast-in place fasteners located in the tidal zone and above. | ASTM A 934 A |
| Hybrid Structures | Structural Steel and Reinforced Concrete Hybrid Structures are allowed | NS 3476 |
| Quality Control | Third Party Continuous Q.A. Inspection and Documentation | ACI 318 |

### 10.3.3 Reinforced Concrete for MOB Construction

#### 10.3.3.1 Scope

To obtain durable concrete, one must carefully consider the materials to be used in the construction. Material characteristics that affect concrete permeability are of particular importance. Prestressed, structural lightweight concrete (LCW) and modified density concrete is the recommended material for the construction of MOB. The scope includes hybrid design concepts employing LCW in combination with structural steel construction.

### 10.3.3.2 Performance History

LWC has a good performance history of producing high-quality concrete for the marine environment.[31, 32] Concrete has been used successfully by the United States for maritime ship and barge construction since 1918 and can perform on an equal basis with comparable steel vessels.[33] "Concrete ships constructed during World War II are still in service and showing little deterioration."[34] "High strength, prestressed, lightweight concrete also offers excellent durability and energy absorption -- two important considerations in harsh environments".[35] Numerous large prestressed LWC marine structures have proven to be durable, including Hibernia, CIDS offshore platform in the Beafort Sea, the Coronado Bridge in San Diego, and the San Francisco-Oakland bay bridge roadway deck. Several platforms in the North Sea have successfully utilised lightweight or modified density concrete.

### 10.3.3.3 Cementitious Materials

The cementitious materials include all materials that chemically bind to form the hardened paste including; cement, fly ash, silica fume and rice husk ash. The cementitious materials must be selected to resist the deterioration mechanisms. Portland cements should comply with ASTM C 150, ASTM C 595, and ASTM C 845. The optimum proportions of ground granulated blast-furnace slag, are 70% and 30% cement. The grind (Blaine fineness) of ground granulated blast furnace slag should be less than 380,000mm$^2$ per/g. (3800 $^{cm^2}/_{g}$). Other cements may be considered to satisfy the performance and durability requirements such as Japanese Belite-Rich or HS 65 Norcem. The use of 5 percent silica fume was used to construct Washington State floating bridges.

### 10.3.3.4 Water-Cementitious Materials Ratio [w/(c+m)]

Durability is directly related to the [w/(c+m)]. It is defined as the ratio of water used to the amount of cementitious materials used in the fresh concrete mixture. The total water content should include the free water on the aggregate but normally excludes that absorbed by the aggregate prior to initial set. Criteria for w/(c+m)] during full-scale production should be

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[31] Holm, T.A. 1980 Performance of Structural Lightweight Concrete in a Marine Environment. ACI Publication, SP, 1-15.

[32] The Federation International de la Precontrainte (FIP) report, State-of-the-Art Report: Lightweight Aggregate Concrete for Marine Structures.

[33] Technical Division of Concrete Control Subsection. 1944. History of the Concrete Ship and Barge Program 1941-1944. U.S. Maritime Commission

[34] Barge-Like Structures ACI Committee Report 357.2R-88 p29. 1988

[35] ACI Committee Report 213B "State-of-the-art Report on Structural Lightweight Aggregate Concrete for Bridges and Other Exposed Structures" Draft October 1996.

established based on the results of the durability study and industry standards.[36] The cementitious materials include all materials that chemically bind to form the hardened paste.

10.3.3.5 Aggregates

The selection of an aggregate source that has a proven record for producing stable and durable materials is essential to meeting the durability requirements for MOB. Lightweight aggregates are typically, processed natural materials yielding concrete with a unit density of less than 120 pcf (1920 kg/m$^3$). Examples include; expanded clay and shale. Use of modified density concrete with a fraction of hard rock aggregate may be considered in order to obtain higher modulus and compressive strength. The ultimate strength and durability properties are highly affected by the characteristics of the aggregate. Important characteristics include; water absorption, water content, uniform strength, stiffness, grading, degree of burning, unit weight, size, surface texture, creep, shrinkage, pore structure, contaminates and manufacturing processes.[37, 38] The compressive strength of the hardened concrete is often limited to the properties of the lightweight aggregate selected. Compressive strengths of 9000 psi (62 MPa) and greater can be reliably produced. ASTM C 330 does not provide sufficient restrictions nor guidelines for lightweight aggregates for marine applications.

It is critically important to select the highest quality aggregate available in order to obtain high performance concrete. Although high-quality lightweight aggregates have been obtained from the U.S. for the Hibernia platform and for major bridges, from Japan for the CIDS, and from Germany for the Troll platform in the North Sea, there are no standards yet established. Critical properties are compression strength, modulus, creep and shrinkage, water absorption and abrasion resistance. In exposures subject to freezing, a maximum of 8% moisture absorption should be required. The lightweight aggregates should not be pre-soaked. Reference is made to research on high-performance lightweight aggregates by the Petroleum Operators' Research Association, as published in the ACI journal. The code and standards requirements for resistance to alkali-aggregate reactivity are not proving adequate for extended marine service beyond 30-50 years. Therefore, the addition of silica in the form of Fly ash and silica fume is important. Petrographic analyses of natural aggregates, both coarse and fine, are recommended.

10.3.3.6 Chemical Admixtures

Admixtures are commonly used to modify the properties of the fresh and hardened concrete. Care should be taken to understand their side-effects and interactions. Their effects are often time and temperature related.[39] Their use with lightweight aggregate concrete may have

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[36] ACI 318R-89

[37] ACI 221R

[38] ACI 213R

[39] ACI 212.3R, Chemical Admixtures for Concrete

different results from those experienced with normal weight materials. Trial batches and sensitivity studies are recommended to optimise the selection and dosages used.

10.3.3.7 Steel Reinforcement (to be written)

10.3.3.8 Prestressed Reinforcement (to be written)

The use of a zero tension under service load criterion should be considered.

10.3.3.9 Fusion-Bonded Epoxy-Coated Steel Reinforcement

The U.S. Navy in co-operation with industry has developed an ASTM Standard for prefabricated fusion-bonded epoxy coated steel reinforcement for use in severe environments.[40] The U.S. Navy has used this technology successfully to construct several piers and the Admiral Clarey Bridge in Pearl Harbor, Hawaii in 1997-98.

The decision to use epoxy-coated rebar in new Navy construction was based on extensive evaluations that began in 1984. The Naval Civil Engineering Laboratory conducted long-term field evaluations at Key West, Florida to rank the relative performance of popular corrosion control methods. Damage-free epoxy-coated rebar performed best.[41] In contrast to these results, some State Highway Agencies experienced a few projects were epoxy-coated rebar performed poorly. Consequently, the Navy identified the failure mechanisms in current practices and drafted new criteria in cooperation with industry experts. The American Society of Testing and Materials (ASTM) used the Navy's draft specifications as a basis for the development of ASTM A 934/A 934M published in July 1995, "Standard Specification for Epoxy-Coated Prefabricated Steel Reinforcing Bars." In February 1998, the NAVFAC Criteria Office published, for the first time, a definitive guide for Marine Concrete, NFGS 03311. Included is a requirement to use prefabricated epoxy-coated reinforcing steel according to ASTM A 934/A 934M .

10.3.4 Considerations for Long-Term Materials Performance

10.3.4.1 Scope

This section presents an introduction to some of the performance relationships that should be considered. To accomplish the durability objective for MOB, emphasis must be placed on understanding the relationships between the environmental conditions, physical loading, deterioration mechanisms and material properties. There are many significant factors involved in these relationships that will ultimately effect the durability of the structure.[42] It is expected that

[40] American Society for Testing and Materials, Standard Specification for Epoxy-Coated Prefabricated Steel Reinforcing Bars, ASTM Designation: A 934/A 934M-97

[41] Concrete Reinforcing Steel Institute Research Series 2, July 1994.

[42] ACI Committee Report 213B "State-of-the-art Report on Structural Lightweight Aggregate Concrete for Bridges and Other Exposed Structures" Chapter 4 Properties of Lightweight Concrete. Draft October 1996.

the designer will review the literature and use the information to aid in formulating the construction criteria.

10.3.4.2 Abrasion Resistance

Abrasion of the concrete surface due to berthing, docking operations, mechanical equipment, cables or ice scouring may wear away the concrete surface. Reinforcement must be placed with sufficient concrete cover to avoid risk of damage or failure. Strength of the cement matrix, bond to the aggregate and hardness of the aggregate must be considered. Considerable data is available because of work done on LWC marine arctic facilities for the oil and gas industry. Relative performance of candidate materials may be evaluated using ASTM C 779.

10.3.4.3 Creep

Excessive creep of the LWC under load can result in cracks and excessive deformation of the structural element. The creep properties of the specified LWC must be determined by laboratory testing since each mixture has unique properties. Creep characteristics are a function of many variables, including the aggregate, mix design, moisture content of the aggregate, curing method, and the ambient humidity and temperature of the exposure site. Creep is accelerated during steam curing and will result in loss of prestress stain.[43]

10.3.4.4 Durability Study

Computer models are being developed by various organizations world wide that are striving to predict the service life of reinforced concrete in the marine environment.[44,45,46,47] Appropriate models should be identified and evaluated for use. The purpose is to use the model to determine what combinations of material properties are needed to produce a prestressed LWC that will perform as required.

10.3.4.5 Crack Control

The matter of crack width control and its impact on corrosion and durability is the most controversial matter in concrete design and construction today. The problem is the exceeding complexity of cracking, the inability to accurately complete and to measure crack widths, the

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[43] Gerwick, Ben. C. "Construction of Prestressed Concrete Structures" Second Edition, Wiley Professional Series, 1993, p. 23

[44] A. Yamamoto, K. Motohashi, S. Misra and T. Tsutsumi, Proposed Durability Design for RC Marine Structures, Concrete Under Severe Conditions Environment and Loading, Volume 1, pp. 544-553. CONSEC '95, 1995

[45] Yokozeki, K., Motohashi, K., Okada, K., and Tsutsumi, T., A Rational Model to Predict the Service Life of RC Structures in Marine Environment SP 170-40 pp.778-799

[46] Collins, F.G., and Grace, W.R., Specifications and Testing for Corrosion Durability of Marine Concrete: the Australian Perspective, SP 170-39 pp. 758-777

[47] Maage, M., Helland, S. and Carlsen, J.E., Service Life Prediction of Marine Structures, SP 170-37 pp.724-743

seemingly chaotic relationship between these and the onset of corrosion. The Norwegian Code NS3473 represents one of the most conservative codes for the amount of reinforcing steel required to control crack width. Excessive use of steel may be counter-productive and expensive.

Variables that affect Crack Widths

- Temperature
- Relative humidity and/or degree of water saturation
- Time (age)
- Orientation and amount of reinforcement
- Cover thickness
- Rate of loading
- Percent of reinforcement transverse to cracks
- Size of bars and spacing
- Number of cycles of loading, thermal extremes and wetting-drying

Variables that affect Corrosion Rate of Reinforcing Steel

- Permeability of concrete cover to oxygen
- Cover thickness
- Moisture and relative humidity, degree of saturation
- Concentration and size of bars
- Stress in the bars

10.3.4.6 Impact Resistance: Impact can best be resisted by use of headed rebars or stirrups in amount of 25% of the through thickness area, in zones of potential impact. (this section hasn't been written yet)

10.3.4.7 Shrinkage: With proper curing, shrinkage strain can be limited to about 400 microstains, ie. $400x10^{-6}$ (this section hasn't been written yet)

10.3.4.8 Corrosion Resistance (this section hasn't been written yet)

10.3.4.9 Fatigue Resistance (this section hasn't been written yet)

10.3.4.10 Fire Resistance Fire resistance is important in interior areas where equipment is operating. High strength lightweight concrete will not provide adequate fire resistance but will spall, probably explosively. Therefore a separate insulating layer of vermiculite or other fire resistant insulating concrete as necessary to obtain the desired resistance to fire.

### 10.3.5 Concrete Materials for Aircraft Traffic Surfaces

#### 10.3.5.1 Scope

Concrete, steel, or a combination of the two may be considered for the MOB aircraft traffic areas. All of the general requirements for concrete, previously stated apply to aircraft traffic surfaces. This section provides supplemental requirements and considerations that are unique to Navy aircraft operations.

#### 10.3.5.2 General Considations

Concrete requires periodic maintenance including crack sealing, patching, and re-sealing of expansion joints. In addition, concrete is subject to the buildup of rubber deposits from landing aircraft that must be removed periodically using high pressure water blasting. Repeated high pressure water blasting will abrade the concrete thickness. Ether 75mm cover or a 37mm additional topping of latex-modified morter., You may want to reference the La Guardia Airport Overwater Runways and Taxiways originally built in 1965 or prestressed hard-rock concrete and under continuous heavy air traffic ever since.

Steel requires a non-skid coating similar to that used on aircraft carriers and landing mats (AM2 mat), that must be re-applied periodically. The designer must also consider the following.

- Stresses and possible damage to the concrete aircraft deck surface due to the flexibility of an underlying steel structure.
- Inspection, maintenance, and repair of concrete
- Frequency, cost, and impacts to operational availability.

#### 10.3.5.3 Applicable Documents

Most all documents for concrete pavement design are based on the premise that the substrate is compacted and stable.[48] The MOB platform may be a relatively flexible surface to construct a concrete aircraft deck upon. Criteria developed for MOB aircraft deck surfaces must be based on simulation from laboratory and field-testing of s floating airfield pavements. The following documents are provided for reference because they contain criteria for aircraft traffic surfaces.

| | |
|---|---|
| • MIL-HDBK-1021/1: | Airfield Geometric Design |
| • MIL-HDBK-1021/2: | General Concepts for Airfield Pavement Design |
| • MIL-HDBK-1021/4: | Rigid Pavement Design for Airfields |
| • NAVFAC DM 21.9: | Skid Resistant Runway Surface |

[48] MIL-HDBK-1021/4. Rigid Pavement Design for Airfields,

- MIL-HDBK-1023/1: Airfield Lighting
- MIL-HDBK-1024/1: Aviation Operational and Support Facilities
- NAVFAC DM 21.06: Airfield Subsurface Drainage and Pavement Design
- NAVFAC DM 2.04: Concrete Structures
- MIL-HDBK-1002/1: Structural Engineering General Requirements
- MIL-HDBK-1002/3: Steel Structures
- NAVFAC P-80: Criteria for Navy and Marine Corps Shore Installations
- ASTM D 2628
- FAA Advisory Circular 150/5320-12C, Measurement, Construction, and Maintenance of Skid-Resistant Airport Pavement Surfaces

10.3.5.4 Concrete

All loading aspects, including static and dynamic that would influence the integrity of the surface to carry the aircraft loading, shall be considered. The fatigue characteristics shall be determined based on the proposed amount of traffic and the combined effect of critical and non-critical design aircraft. The concrete shall perform as the wearing surface for aircraft traffic and meet the following draft criteria.

Table 10.2 Draft Criteria for Concrete Aircraft Surfaces

- Maximum longitudinal slope of 0.5 percent.
- Transverse crown with slope no less than 1 percent or greater than 1.5 percent.
- Skid resistance within acceptable limits of > 0.7 ($M_u$ number) per U.S. Navy and Federal Aviation Administration (FAA) Advisory Circular AC No. 150/5320-12C dated 3.18.97, Measurement, Construction, and Maintenance of Skid-Resistant Airport Pavement Surfaces.
- Roughness index such that resonance frequency of the aircraft is avoided for all anticipated aircraft and landing situations. Acceptable Roughness is a function of wheel spacing, mass of the aircraft, speed, landing gear suspension, and other parameters.
- The concrete shall be maintainable according to the Condition Indexes (PCI) and not fall below the value of 70 for the runway and 60 for all other areas subjected to aircraft traffic.
- Shall be structurally adequate for all loading conditions including but not limited to aircraft static and dynamic loads, and blast and high temperature exhaust effects.
- Shall be resistant to the combined effects of fatigue and differential movements.
- Shall be resistant to abrasion of wheel loads and arresting gear.
- Concrete reinforcement can be ordinary deformed reinforcing bars, prestressed, and/or post-tensioned reinforcing strand.
- Fusion-bonded epoxy-coated steel reinforcement shall not be used in the runway.
- Supplemental reinforcement such as synthetic fibers are allowed.
- The surface shall not contain any steel fiber reinforcing material
- Longitudinal Joints - use Preformed Polychloroprene Elastomeric Joint Seal
- Transverse Joints - use Dow Corning 890-SL Self-Leveling Silicone Joint Sealant (or equivalent) with a compatible backer rod as recommended by the manufacturer.
- Airfield Marking Paints - as recommended in NAVFAC Guide Specification 02761A.
- Aircraft Tiedowns - as recommended in NAVFAC Guide Specification 02762A and MIL-HDBK-1021/4
- Arresting Gear Inlays: Use NAVFAC Definitive Steel Plate Design (NAVFAC definitive drawings 1404521 and 1404522)
- VSTOL Launch Pads: Use AM-2 Matting
- Parking Areas Subject to Jet Aircraft Auxiliary Power Units: Place an inlay of high performance concrete resistant to blast and high temperature exposures. Proprietary materials such as "Set 45" cement used with a lightweight aggregate such as "Solite" has been evaluated by the Naval Facilities Engineering Service Center as having enhanced properties compared to conventional concrete mixtures. Other surfaces resistant to blast and high temperature exposure such as steel plates or AM-2 Matting may be used.

### 10.3.6 Quality Control

The literature recommends that strict adherence to quality control is necessary to assure that good concrete placement practices are followed. The contract must contain necessary quality control requirements to assure that the repairs are accomplished satisfactorily.

10.3.6.1 Third Party Inspection:

The Navy will hire a third party certified Quality Assurance (Q.A.) firm to assure that the highest quality product is consistently produced for MOB construction. The Q.A. firm must have qualified experience and a full time licensed engineer. Daily Q.A. documentation must be submitted to the Contract Officer for review.

10.3.6.2 Demonstration:

The contractor shall successfully demonstrate to the Navy a successful method of forming, placing, prestressing, and curing the concrete prior to full-scale production. All appropriate parties should witness and approve the materials, procedures and quality control program. The demonstration will set the quality standard for the full-scale production.

10.3.6.3 Laboratory Testing

10.3.6.4 Field Controls

Aggregate
Mixing
Placement
Unit Weights
Air Content
Finishing
Curing

----------------- end ------------------------

# PART C

# PATCH REPAIRS

This section contains guidelines, specifications, and case studies for marine concrete repair above and below the waterline.

# CHAPTER 7

# NFESC SPECIFICATIONS FOR MARINE CONCRETE REPAIR

# NFESC

## SPECIFICATIONS FOR:

## Marine Concrete Repair

## For:

**Top deck patch repair**
**Under deck hand pack patch repair**
**Drip edge for penetrations**
**Crack repair**
**Joint repair**

# MARINE CONCRETE REPAIR
## SITE DEMOLITION

**1. GENERAL**

1.1 REFERENCES

The publications listed below form a part of this specification to the extent referenced. The publications are referred to in the text by the basic designation only.

AMERICAN NATIONAL STANDARDS INSTITUTE (ANSI)
CODE OF FEDERAL REGULATIONS (CFR)

1.2 GENERAL REQUIREMENTS

Do not begin demolition until authorization is received from the Contracting Officer. Continuously remove rubbish and debris from the project site; do not allow accumulations on the structure deck. Store materials that cannot be removed daily in areas specified by the Contracting Officer. Contain all materials from entering the harbor waters.

1.3 SUBMITTALS

Submit the following documentation to the Contracting Officer prior to receiving authorization to proceed with demolition.

1.3.1 Statements

a. Demolition plan
b. Notification of demolition and renovation

Submit proposed demolition and removal procedures to the Contracting Officer for approval before work is started.

1.3.1.1 Required Data

Demolition plan shall include procedures for coordination with other work in progress, a detailed description of methods and equipment to be used for each operation and of the sequence of operations.

1.4 REGULATORY AND SAFETY REQUIREMENTS

Comply with federal, state, and local hauling and disposal regulations. In addition to the requirements of the "Contract Clauses," safety requirements shall conform with ANSI A10.6, "Demolition Operations - Safety Requirements."

1.4.1 Notifications

Furnish timely notification of demolition and renovation projects to Federal, State, regional, and local authorities in accordance with 40 CFR 61-SUBPART M, if required. Notify the local air pollution control district/agency and the Contracting Officer in writing 10 days prior to the commencement of work in accordance with 40 CFR 61-SUBPART M.

1.5 DUST AND DEBRIS CONTROL

Prevent the spread of dust and debris and avoid the creation of a hazard or nuisance in the surrounding area. Do not use water if it results in hazardous or objectionable conditions such as, but not limited to, pollution or runoff into the harbor waters. Minimize the application of water to that required for dust control. Prevent unpermitted discharges into the storm sewer, soil and harbor. Water washdown of the areas is not allowed. Prevent all debris concrete cutting, chipping, demolition, and repair from entering the harbor waters. The contractor must retrieve any material that enters harbor waters.

### 1.6 PROTECTION

#### 1.6.1 Traffic Control Signs

Where pedestrian and driver safety is endangered in the area of removal work, use traffic barricades with flashing lights. Notify the Contracting Officer prior to beginning such work.

#### 1.6.2 Existing Work

Protect existing work that is to remain in place, be reused, or remain the property of the Government. Repair items which are to remain or which are to be salvaged that are damaged during performance of the work to their original condition, or replace with new. Do not overload structural elements. Additional structural supports and reinforcement must have Contracting Officer approval.

#### 1.6.3 Facilities

Protect electrical and mechanical services, utilities, and facilities. Where removal of existing utilities and pavement is specified or indicated, provide approved barricades, temporary covering of exposed areas, and temporary services or connections for electrical and mechanical utilities.

### 1.7 BURNING

Burning will not be permitted.

## 2. EXECUTION

### 2.1 EXISTING FACILITIES TO BE REMOVED

#### 2.1.1 Concrete

Break out the concrete and prepare repair surface as detailed in the Contract Drawings and the Specifications for REPAIRS.

#### 2.1.2 Demolition

Prior to the start of demolition or crack repairs, the areas to be demolished or repaired shall be marked out and jointly inspected by the Contractor, Contracting Officer and quantities estimated as specified under the Specifications for DECK REPAIRS and SILICONE CRACK REPAIR. Abandoned pipelines and conduits may be removed at the discretion of the contractor to facilitate concrete repairs.

### 2.2 TITLE TO MATERIALS

Except where specified in other sections, all materials and equipment removed, and not reused, shall become the property of the Contractor and shall be removed from Government property. Title to materials resulting from demolition, and materials and equipment to be removed, is vested in the Contractor upon approval by the Contracting Officer of the Contractor's demolition and removal procedures, and authorization by the Contracting Officer to begin demolition. The Government will not be responsible for

the condition of, loss of, or damage to, such property after contract award. Materials and equipment shall not be viewed by prospective purchasers or sold on the site.

2.3 CLEANUP

2.3.1 Debris and Rubbish

Remove and transport debris and rubbish in a manner that will prevent spillage into the ocean, harbor or bay waters, on streets, or adjacent areas. Clean up spillage from the structure and adjacent areas daily.

# CONCRETE REPAIRS
## DECK REPAIRS

1. **GENERAL**

This specification covers the use of prepackaged cementitious concrete repair materials and procedures for making partial-depth repairs to Structure.

1.1 REFERENCES

The publications listed below form a part of this specification to the extent referenced. The publications are referred to within the text by the basic designation only.

AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS (AASHTO)

| | |
|---|---|
| AASHTO T 2771989 | Standard Method of Testing for Rapid Determination of the Chloride Permeability of Concrete |

AMERICAN CONCRETE INSTITUTE

| | |
|---|---|
| ACI 301 (1994) | Structural Concrete for Buildings |

AMERICAN SOCIETY FOR TESTING AND MATERIALS (ASTM)

| | |
|---|---|
| ASTM C 309 (1994) | Liquid Membrane-Forming Compounds for Curing Concrete |
| ASTM A 615 | 1993 Deformed and Plain Billet-Steel Bars for Concrete Reinforcement |
| ASTM C 33 | 1993 Concrete Aggregates |
| ASTM C 109 | 1991 Standard Method for Compressive Strength of Hydraulic Cement Mortars |
| ASTM C 157 | Standard Test Method for Length Change of Hardened Hydraulic-Cement Mortar and Concrete |
| ASTM C 490 | Standard Practice for Use of Apparatus for the Determination of Length Change of Hardened Cement Paste, Mortar, and Concrete |
| ASTM C 496 | 1990 Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens |
| ASTM C 882 | 1991 Test Method for Bond Strength of Epoxy-Resin Systems Used with Concrete (modified for cementitious material) |
| ASTM C884 | 1987 Test Method for Thermal Compatibility Between Concrete and an Epoxy-Resin Overlay |

1.2 DESCRIPTION OF WORK

Concrete repair work must be accomplished prior to installing the cathodic protection system and the application of composite upgrade materials. The concrete repair work consists of several parts: A) Partial-depth repairs of deteriorated concrete on the top and bottom of the structure deck. B) Removal of steel crane rail and placement of concrete into the resulting cavity. C) Removal of sound concrete that protrudes from the under deck surfaces. These areas are identified as existing "built-up" repairs. Removal of the built-up areas may result in a cavity, which must be repaired with concrete to establish a suitable surface profile flush with the adjacent surfaces. D) Removal of sound concrete along some of the construction joints on the underside of the deck to establish a suitable surface profile flush with the adjacent surfaces. E) Installation of a drip edge at existing holes on the underside of the deck to direct water away from the concrete surface. F) Sealing construction joints and cracks on the top of the deck to prevent water from wetting the underside of the deck. The repair work shall proceed by removing concrete from the areas identified by the contracting officer using approved methods identified in the Contract Drawings and herein, cleaning the area by abrasive blasting, placing an approved bonding agent, placing an approved repair material, finishing and texturing, curing, and, finally, sealing joints and saw overcuts.

## 1.3 LOCATION

The Contracting Officer will designate the locations and boundaries of each repair area with the Contractor. The Contractor will remove all unsound concrete and expose the rebar as necessary based on the repair criteria so that no visible corrosion is evident beyond normal "mill scale." Refer to Contract Drawings.

## 1.4 SUBMITTALS

Submit the following documentation and materials to the Contracting Officer prior to receiving authorization to proceed with concrete repair. Some laboratory tests shall be conducted on 35-day-old specimens. Therefore, it would be prudent for the Contractor to prepare for these tests with an approved test laboratory well in advance to avoid delays in the concrete repair work.

### 1.4.1 Manufacturer's Catalog Data/Instructions

a. Cementitious Repair Material
b. Curing Compounds

### 1.4.2 Laboratory Test Results and Verification

The Contractor will submit to the Contracting Officer test results from an approved concrete laboratory showing that the repair material meets or exceeds the Navy's specifications on shrinkage and strength. Some laboratory tests shall be conducted on 35-day-old specimens. Therefore, it would be prudent for the Contractor to prepare for these tests well in advance to avoid delays in the concrete repair work.

### 1.4.3 Batch Samples

When requested by the Contractor shall provide batch samples of any materials that are used throughout the progression of the work. Batch samples will not exceed 2% of the total material used on this job.

## 1.5 DELIVERY, STORAGE, AND HANDLING

Inspect materials delivered to site for damage, unload and store with a minimum of handling. Deliver cementitious repair material components and aggregate materials in original sealed containers and store in dry covered areas at temperatures below 100°F.

## 1.6 WEATHER LIMITATIONS

Halt work when the weather conditions are inclement and detrimentally affect the quality of patching concrete. Windy conditions and rain will affect the concrete curing. Apply patching materials only when the atmospheric and surface temperature ranges are suitable for the specified material. Halt work if the

temperature is below 40°F (4°C). Follow manufacturer's instructions for weather conditions and temperature ranges. Patches placed during adverse weather conditions may have to be removed and replaced.

1.7 EQUIPMENT

Use a container recommended by the manufacturer as the mixing vessel. Use equipment specified by repair material manufacturer for field mixing, transporting and consolidation of cementitious repair materials.

1.8 QUALITY ASSURANCE

A Technical Representative of the manufacturer of the cementitious repair material being used shall be present during the start of repair work. The Technical Representative shall inspect and approve the surface preparation and observe the initial application. The Technical Representative may demonstrate and instruct the Contractor on proper procedures.

A written report shall be submitted to the Contracting Officer outlining the Technical Representative's observations and suggestions, including but not limited to recommendations regarding mixing and placement procedures, and equipment used for mixing, placement, consolidation, and curing.

Mixing and handling instructions shall be available at the site at all times during the repair operations.

Throughout the progress of the work in this Specification, the Contractor will provide at least one (1) person certified by the material manufacturer who is thoroughly familiar with the specified requirements, completely trained and experienced with the necessary skills, and who will be present on the site and direct all work performed under this Specification.

In performing the work of this Specification, the Contractor will use an adequate number of skilled workmen to ensure the installation is in strict accordance with schedule, specification and the procedures recommended by the material manufacturer.

**2. MATERIALS**

2.1 MATERIAL SPECIFICATIONS

The materials used shall meet the requirements of the following specifications as well as other Contracting Officer approved Proprietary repair materials:

| | |
|---|---|
| AASHTO M-80 & M-6 | Aggregate |
| AASHTO M-148 | Curing compound |
| AASHTO M-194 | Concrete admixtures |

2.1.1 Cementitious Patch Material

The product shall be prepackaged by the manufacturer with premeasured, properly proportioned components. It shall be suitable for the hand-packed repair method (see Contract Drawings) and shall have the following properties:

a. Minimum pot life of 30 minutes at 75°F.

b. Bond strength per ASTM C 882 modified for cementitious material at 28 days: 2,200 psi minimum.

c. Maximum permeability of 1,000 coulombs per AASHTO T 277.

d. Drying shrinkage: Specimens shall be prepared per ASTM C 157 as modified to use molds per ASTM C 490 (3x3x11.25 inches) with a 10-inch gauge length. During the first 7 days the molded specimen shall be covered with a water-saturated rug or burlap. After the 7 day wet curing period, the mold shall be removed and the specimen cured for an additional 28 days at 46 to 54% relative humidity at 70 to 76°F. The ultimate shrinkage to be reported is that value measured at the end of the 35$^{th}$ day. Allowable shrinkage shall not exceed 0.05%.

e. Minimum compressive strength per ASTM C 109 modified for cementitious material shall be 3,000 psi @ 3 days.

f. The water to cementitious ratio shall not exceed 0.40.

2.1.2 Aggregate

If aggregate is added to the prebagged mixture, then all tests for acceptance criteria per Section 2.1.1 shall be conducted with the added aggregate. Aggregate added to the repair material, if allowed by the manufacturer, shall be 3/8-inch minus, clean, well graded, saturated surface dry material, having low absorption and high density, and conform to ASTM C 33. Aggregate must be approved for use by the Contracting Officer.

2.1.3 Reinforcing Bars

ACI 301 unless specified otherwise. ASTM A 615, Grade 60 bars.

2.1.4 Laboratory tests per Section 2.1.1 shall be submitted to the Contracting Officer for approval before concrete repair can proceed. Laboratory tests shall be conducted on 35-day-old specimens. Therefore, it would be prudent for the Contractor to prepare for these tests well in advance to avoid delays in the concrete repair work.

## 3. EQUIPMENT

### 3.1 GENERAL

The Contractor shall furnish and maintain such equipment as necessary to complete the work in accordance with the specifications.

### 3.2 CONCRETE SAW

The concrete saw shall be equipped with a diamond blade(s) or approved equal. The saw shall be capable of sawing concrete to the specified depth without damaging the surrounding concrete. Depth of cut shall be adjusted so as to avoid cutting the existing steel reinforcement.

### 3.3 CONCRETE REMOVAL EQUIPMENT

The Contractor shall provide equipment capable of removing the deteriorated concrete in the repair area to the depth required without damaging the sound concrete surrounding or below the repair. The Contractor shall provide the necessary means to assure that no concrete debris or slurry water enters the harbor waters, refer to SITE DEMOLITION Specifications.

3.3.1 Pneumatic Jackhammers

Jackhammers heavier than 15 pounds (6.8 kg) shall not be permitted.

3.3.2 Abrasive Blasting or Mechanical Scarification

Abrasive blasting or mechanical scarification shall be capable of removing all contaminants and loose particles from the surface of the steel reinforcement and concrete in the repair area. The equipment shall be fitted with suitable traps, filters, drip pans, or other devices to prevent oil, fuel, grease, or other undesirable matter from being deposited on the cleaned surface and the harbor waters.

3.3.3 Brooms, Shovels

Stiff-bristled brushes shall be used to apply the bonding agent. Shovels may be used to place the repair materials, if appropriate.

3.4 FINISHING AND FLOATING EQUIPMENT AND STRAIGHTEDGES

The finishing and floating equipment shall be capable of consolidating and floating the concrete. A dense, homogenous repair must be produced and finished to the same surface slope as the existing concrete slab.

3.4.1 Pressure Hand Sprayer for Membrane-Curing Compounds

The pressure sprayer for membrane-curing compounds shall be capable of providing a uniform, even coating of the compound over the surface of the repair. Manually operated spray equipment may be used.

## 4. CONSTRUCTION METHOD

4.1 DETERMINATION OF REPAIR AREAS

The Contracting Officer shall determine areas to be repaired by using a hammer or other techniques to determine the extent of the unsound concrete. The Contracting Officer shall mark the boundaries of the repair area. Large areas such as the rail slot may use flowable repair materials while small areas and all areas below deck shall be repaired by the dry-pack method. Holes through the deck will be either filled with low shrinkage concrete or lined with a drip edge according to the Contract Drawings. All previous built-up repairs under the deck that interfere with areas to be structurally upgraded must be modified to achieve a compatible surface profile with the adjacent concrete surfaces. See the Contract Drawings for details of repairs.

4.2 PREPARATION OF REPAIR AREA

A hand-held 15-lb chipping hammer may be used. All other methods must be approved by the Contracting Officer.

4.2.1 Concrete Removal

The deteriorated material in the repair area shall be removed using the methods specified in this section. A saw cut shall be made around the perimeter of the repair area to provide a vertical face at the edges and sufficient depth for the repair. The saw cut shall have a minimum depth of 1 inch (25 mm). Depth of cut shall be selected to preclude cutting reinforcing steel bars.

Concrete within the repair area shall be broken out to a minimum depth of 2 inches (51 mm) or until sound concrete is exposed.

Remove loose concrete from the designated areas. Inspect the cavity for remaining unsound concrete by tapping with a hammer or steel rod. In areas where tapping indicates unsound concrete, remove additional concrete. Make the entire cavity at least 2 inches deep. Where rebar is exposed remove all corrosion by abrasive blasting or mechanical means to a near white metal condition as per recommendations of patch material manufacturer, prior to installing patch material. Continue to “chase” all corroded steel

reinforcement until no corrosion is visible beyond normal "mill scale." Prepare surfaces by abrasive blasting or mechanical scarification to achieve a uniformly rough surface.

4.2.2 Concrete Removal of Built-up repairs (underside of deck)

Existing form and pump concrete repairs made to the underside of the deck were built up to increase the cover over the steel, resulting in areas that are not flush with the adjacent concrete surface. Typically these built-up areas extend 2 to 3 inches from the original surface. These areas must be cut back, so that they are flush to permit the application of the structural upgrade materials. The contractor shall propose to the Contracting Officer the method by which the concrete will be removed. If a cavity results from the removal of the concrete, then this area must be repaired so that the surface is flush with the adjacent concrete.

4.2.3 Hand-Held Chipping Hammer

Concrete within the repair area shall be broken out to a minimum depth of 2 inches (50 mm) with pneumatic tools until sound concrete is exposed. The maximum size pneumatic hammer shall be 15 pounds (6.85 kg). Pneumatic hammers and chipping tools shall not be operated at an angle exceeding 45 degrees from the vertical. Such tools may be started in the vertical position but must be immediately tilted to a 45-degree operating angle. The removal shall start within the interior of the repair and work outward. Care shall be used to prevent fracture of the sound concrete below the repair area and the surrounding concrete. A minimum 1-inch (25-mm) vertical face (saw cut) on all sides shall be provided. However, adjustments shall be made to avoid cutting any steel rebar. All concrete chips/debris shall be contained and prevented from falling into the harbor waters.

4.3 SURFACE PREPARATION

4.3.1 Concrete

Abrasive blast or mechanically scarify the exposed faces of the concrete to remove all loose particles, oil, dust, cement or slurry residue, paint, and other contaminants. Immediately prior to placing the concrete bonding agent, clean the exposed surfaces by compressed air blasting. All loose particles, oil, dust, cement or slurry residue, paint, and other contaminants shall be contained and prevented from falling into the harbor water.

4.3.2 Steel Reinforcement

Reinforcing steel bar that has lost more than 25% cross-sectional area must be repaired by welding a new segment of rebar of the same diameter to the existing rebar. Corroded or damaged rebar will be identified in the field by the contractor and verified for replacement by the Contracting Officer. The splice will cross the damaged length and the welds made at locations where the existing rebar is in excellent condition without loss of area. New reinforcing steel shall be ASTM A-615 grade 60 and welded in accordance with the Structural Welding Code – Reinforcing Steel (AWS D1.4). The welding surface shall be prepared by power cleaning as per SSPC-SP11. The weld will be a continuous 0.25-inch fillet that is at least 2 inches long. The contractor will remove any concrete that is damaged during the welding process. Abrasive blast or mechanically clean the steel to bright steel no more than 48 hours prior to application of concrete patch material. All loose particles, oil, dust, cement or slurry residue, paint, and other contaminants shall be contained and prevented from falling into the harbor water.

4.4 APPLYING THE BONDING AGENT

Use a bonding agent recommended by the supplier of the repair material. It may consist of neat cement, cement-sand, or latex-cement slurry. Bonding agents must be approved by the Contracting Officer. Apply the bonding agent to a clean surface saturated dry (SSD) concrete substrate and scrub it into the surfaces using a stiff-bristled brush. Bonding agents that contain epoxy will not be allowed.

### 4.5 PLACING THE REPAIR MATERIAL

Always place materials containing aggregate with a shovel to avoid segregation. Flowable materials may be placed by a bucket or other suitable means.

#### 4.5.1 Proprietary Repair Materials

Place dry pack repair materials according to the method in the Contract Drawing. The application shall be in accordance with manufacturer's recommendations. Special attention shall be paid to pack the material below reinforcing bars and to working the material into the concrete substrate to achieve a sound bond. Use a hard wood dowel to ram the material tightly below and around reinforcing.

### 4.6 FINISHING REQUIREMENTS

Partial-depth repairs are usually small enough so that a stiff board resting on adjacent sound concrete can be used as a screed. Work the materials toward the perimeter of the patch to establish contact and enhance bonding to the existing slab. Make at least two passes with the screed to ensure a smooth repair surface that is level with the surface of the deck. Care should be taken to not "overwork" the surface.
When practical, match the surface texture of the repair with that of the surrounding deck.

### 4.7 CURING

Spray apply two coats of a concrete curing compound (ASTM C 309) as soon as the concrete surface has set sufficiently to apply the curing agent without damage. Apply the curing compound at the rate of 150 $ft^2$/gal (3.7 $m^2$/L). In addition, repairs to the top deck shall also be moist cured for 7 days by covering with saturated pieces of wet rug or carpet.

### 4.8 SAW OVERCUTS

The saw cuts extending from the repair area into to the surrounding sound concrete must be filled with epoxy mortar or cement mortar.

### 4.9 OPENING TO STRUCTURE OPERATIONS

The concrete repairs may be opened to structure operations when a compressive strength of 3,000-psi (21 MPa) has been achieved.

## 5. QUALITY CONTROL

### 5.1 CONCRETE REPAIR MATERIALS

Material supplied to the job shall comply with Section 2.1.1 of this specification. Test reports shall be from an independent testing laboratory approved by the Contracting Officer.

### 5.3 INSPECTION

The Contracting Officer shall check each repaired area for cracks, spalls, popouts, and loss of bond between repaired area and surrounding concrete one week after the repair material was placed. Each repair area will be checked for voids by tapping with a hammer. In addition, they may take one, 1-inch diameter core in each span to verify depth, bonding integrity, and material quality of the concrete repair. The Contractor shall repair the cored site to the same level as required by this specification. Areas found to be defective will be removed and replaced by the Contractor to the satisfaction of the Contracting Officer and the required performance and quality level of this specification.

# CONCRETE REPAIR
## CRACK SEALANT

This specification applies to the requirements for repairing either static or dynamic concrete cracks by sealing on the top surface of the deck. This specification details both application procedures and material requirement. The sealant must provide a service life for a minimum of 10 years in a marine environment.

**1. GENERAL**

1.1 REFERENCES

The publications listed below form a part of this specification to the extent referenced. The publications are referred to within the text by the basic designation only.

AMERICAN SOCIETY FOR TESTING AND MATERIALS (ASTM)

| | |
|---|---|
| ASTM D 412 | 1997 Standard Test Method for Vulcanized Rubber and Thermoplastic Rubbers and Thermoplastic Elastomers - Tension |
| ASTM D 638 | 1996 Standard Test Method for Tensile Properties of Plastics |
| ASTM D 1475 | 1990 Standard Test Method for Density of Paint, Varnish, Lacquer, and Related Products |
| ASTM D 2240 | 1997 Standard Test Method for Rubber Property - Durometer Hardness |
| ASTM D 5893 | 1996 Standard Specification for Cold Applied, Single Component, Chemically Curing Silicone Joint Sealant for Portland Cement Concrete Pavements |

CODE OF FEDERAL REGULATIONS (CFR)

| | |
|---|---|
| 29 CFR 1910.134 | Respiratory Protection |
| 29 CFR 1926.59 | Hazard Communication |
| 40 CFR 261 | Identification and Listing of Hazardous Waste |

1.2 SUBMITTALS

Submit the following documentation and materials to the Contracting Officer prior to receiving authorization to proceed with crack sealing. The Contractor shall use a silicone sealant. The color of the sealant must be gray.

1.2.1 Instructions

a. Two Part Self-Leveling Silicone Joint Sealant

Submit formulator's printed instructions to include brand name, catalog numbers, and names of manufacturers. Include in the instructions detailed mixing and application procedures, quantity of material to be used per size of crack, total quantity of material to be used on job, minimum and maximum application temperatures, and curing procedures. Include copies of Material Safety Data Sheets (MSDS) for all materials to be used at the job site in accordance with OSHA & 29 CFR 1926.59.

1.2.2 Field Test Reports

a. Tests and Inspections

The contractor will submit reports on tests and inspections as set forth in section 3.3.

1.2.3 Certificates

a. Two Part Self-leveling Silicone Joint Sealant

Certify conformance to the requirements set forth in section 2.1.1.

b. Primer Coat Material

Certify conformance to the requirements set forth in section 2.1.2.

c. Bond Breaker Material

Certify conformance to the requirements set forth in section 2.1.3.

1.2.4 Records

a. Installers Qualification.

The Contractor shall have verifiable and specific experience repairing and sealing concrete cracks and expansion joints utilizing routing equipment, bondbreaker tape, and silicone sealant. Throughout the progress of the work in this Specification, the Contractor will provide at least one (1) person who is thoroughly familiar with the specified requirements for crack sealing with silicone, certified by the material manufacturer to be completely trained and experienced with the necessary skills, and who will be present on the site and direct all work performed under this Specification.

In performing the work of this Specification, the Contractor will use an adequate number of skilled workmen to ensure the installation is in strict accordance with schedule, specification and the procedures.

b. Disposal of Material.

All unused material, whether in its cured or uncured state, shall be removed from the job site by Contractor. No material will be allowed to enter harbor waters. Any material that enters harbor waters will be retrieved by the Contractor.

1.2.4 Batch Samples

When requested by the Contractor shall provide batch samples of any materials that are used throughout the progression of the work. Batch samples will not exceed 2% of the total material used on this job.

1.3 DELIVERY AND STORAGE

Ship sealants and other materials in their original, sealed containers. Materials delivered to site shall be inspected for damage and container opening prior to use. Material delivered in dented, rusty, leaking, or previously opened containers and, in addition, material with an expired shelf life shall be returned to manufacturer. Material shall be unloaded and stored out of sun and weather, preferably in air-conditioned spaces.

1.4 SAFETY

Ensure that employees are trained in the requirements of OSHA &29 CFR 1926.59 and understand the information contained in the MSDS for their protection against toxic and hazardous chemical effects. Follow safety procedures as recommended by manufacturer. Procedures may include employing the use of impervious clothing, gloves, face shields, and other appropriate protective clothing necessary to prevent eye and skin contact with materials.

## 2. PRODUCTS

### 2.1 MATERIALS

#### 2.1.1 Silicone Joint Sealants

Silicone joint sealants shall be rapid cure, 100 percent silicone, self-leveling, two-part formulation, and cold applied. Acid cure sealants are not acceptable for use on concrete. Silicone sealant shall be compatible with the surface to which it is applied.

Rapid cure is defined as developing sufficient integrity within 8 hours to accommodate both thermal and/or vertical movements due to traffic loading.

| | |
|---|---|
| Specific Gravity, as per ASTM D 1475 | 1.25-1.35 |
| Nonvolatile Content (% minimum) | 93 |
| Skin-over Time (minutes, maximum) | 20 |
| Joint Elongation (% minimum), as per ASTM D 5893<br>Section 14 modified – pull rate (2 in./min.) and joint size<br>Joint size = ½" x ½" x 2" | 600 |
| Joint Modulus (psi, @ 100% elongation), as per ASTM D 5893<br>Section 14 modified – pull rate (2 in./min.) and joint size<br>Joint size = ½" x ½" x 2" | 3-12 |

#### 2.1.2. Primer Coat Material

Once the crack has been cut, cleaned and dried, the crack shall be coated with a primer prior to the installation of the bond breaker and sealant. The primer coat material to be used shall be from the same manufacturer as that of the silicone sealant and as recommended by the sealant manufacturer and must be compatible with the concrete, bond breaker, and the sealant.

#### 2.1.3. Bond Breaker Material

A bond breaker material shall be installed prior to installation of the sealant.

##### 2.1.2.1. Purpose of Bond Breaker

a. Maintain minimum and/or maximum depth of sealant.
b. Prevent three (3) sided adhesion of sealant. Bond breaker serves to ensure that the bottom of the sealant is bond free thereby allowing sealant to adhere to the sides of the joint only.

##### 2.1.2.2. Requirements of a Bond Breaker

a. Shall be compatible with sealant or any component of the joint sealant system.
b. No bond or adverse reaction shall occur between the bond breaker and the sealant.

2.1.2.3. Acceptable Types of Bond Breakers

a. Closed-cell expanded polyethylene foam backer rod. Primary use is with new joint construction and uniform remedial joint construction.
b. Bond breaker tape or approved equal. Primary use is with wide, shallow joints.
c. Backing material that is open cell with an impervious skin to prevent adhesion. Primary use is with irregular remedial joint construction.

## 3. EXECUTION

### 3.1 CRACK PREPARATION

3.1.1 Locate Rebar and Conduit

All rebar and conduit located a minimum distance of three inches from each side of the crack shall be identified and mapped along the length of the crack. Mapping shall include depth of concrete cover and the exact location of rebar and/or conduit in relation to the crack. Cracks shall be 1/4" wide by 5/8" to 1" deep. The backer rod diameter shall be 3/8" with the sealant thickness of 1/4" (even with the top surface). Joints shall be cleaned and prepped by removing the existing sealant and using an abrasive blast to clean the surface of the joint. The following procedures shall be observed for both cracks and joints.

3.1.2 Surface Cleaning

Cracks shall be routed/cut out to 1/4" width and 5/8" to 1" depth without disturbing or damaging existing rebar and conduit. All dirt, debris, efflorescence, chipped concrete, grease, oil, and other obtrusive material in each crack shall be removed both inside and a minimum of one half inch (1/2") in width on both sides of each crack to be repaired. Cleaning shall be accomplished by a combination of wire brushing, hand tool cleaning, power tool cleaning, compressed air, and aqueous based detergent cleaning. Cleaning utilizing organic solvents is prohibited. All dirt, debris, chipped concrete, grease, oil, and other obtrusive material removed shall be contained and prevented from falling into the harbor water.

3.1.3. Primer Coat

Use a primer recommended by the sealant manufacturer. Only apply the primer following the day's high temperature to avoid off gassing of the concrete and to permit maximum penetration of the primer. Prepare the joint by abrasive blasting and then air blowing the joint. Air compressors used for this purpose must be equipped with traps capable of providing moisture and oil free air. As with any application involving primer, backer material should not be installed until the primer is applied to avoid pooling of primer at the joint wall interface.

For best results, apply the primer with a mist sprayer. (Applying with a clean, lint-free cloth is an alternative, although less desirable method.) Uniformly coat the entire surface primer, being careful not to saturate the substrate. If applied correctly, the substrate will darken in appearance, but there should be no signs of primer run down. Once dry, there should be no visible signs of the primer, only a slight odor. If the primer is over-applied, a white powder will form the substrate surface. If this occurs, the joint/crack must be recleaned and the process repeated.

Allow the primer to dry for approximately 60 minutes. Gently air blow the prepared joint/crack, install the backer material and then install the joint sealant as recommended.

3.1.4 Install Bond breaker

The bond breaker shall be applied to the inner base of the routed out, cleaned, and dry crack. Bond breaker shall be placed flush with inner walls of crack.

## 3.2 SEALANT APPLICATION

### 3.2.1 Mixing

Based on ambient temperature, relative humidity, and moisture content in concrete, consult sealant manufacturer and mix silicone sealant components in accordance to their recommendations.

### 3.2.2 Sealant Installation

Immediately following the day's high temperature and on dry concrete, pour into crack the silicone sealant over the polyethylene bond breaker tape and finish by trowel. Resulting crack repair shall be flush with the surrounding concrete, exhibit complete crack depth penetration, and be free of surface irregularities, air voids, and discontinuities greater than 1/32 inch. A primer shall be used to aid adhesion on either questionable concrete or concrete that contains excess moisture. Primer shall be chemically and mechanically compatible with silicone sealant.

### 3.2.3 Curing

Within forty-eight hours following application of sealant, sealant shall be tack free and ready for light traffic. If after forty-eight hours, the sealant is tacky or in any form of its uncured state, all uncured material shall be removed by Contractor. New sealant will be reapplied to the crack.

## 3.3 FINAL INSPECTION

Government shall inspect and verify repairs have been carried out in accordance with the guidelines set forth in sections 3.2.2 and 3.2.3. The contractor shall take one, 1-inch diameter core on each span to verify depth and quality of sealant penetration. The contractor will repair the cored site to the same level as required by this specification.

## 3.4 FINAL CLEANUP

Following completion of work, remove debris, equipment, and materials from the site. Remove temporary connections to Government furnished services. Restore existing facilities in and around the work areas to their original condition.

# CHAPTER 8

# PLACEMENT DETAILS

GENERAL NOTES:

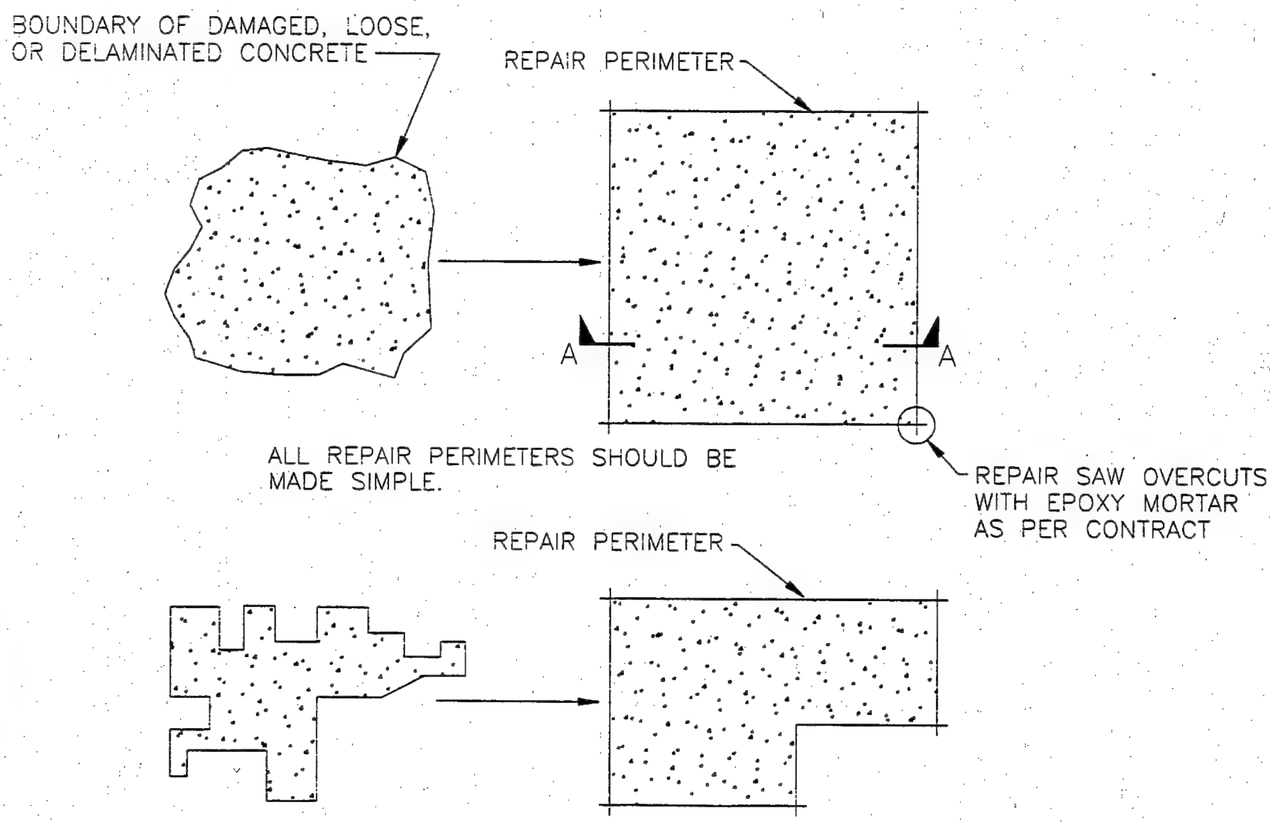
1. ALL CONCRETE REPAIRS ARE OF PARTIAL DEPTH AND SHALL NOT GO THROUGH THE ENTIRE DECK THICKNESS.

2. ALL DEBRIS, ABRASIVE BLASTING GRIT MATERIAL, SAW CUT CEMENT POWDER, AND SLURRY SHALL NOT BE ALLOWED TO ENTER THE WATER.

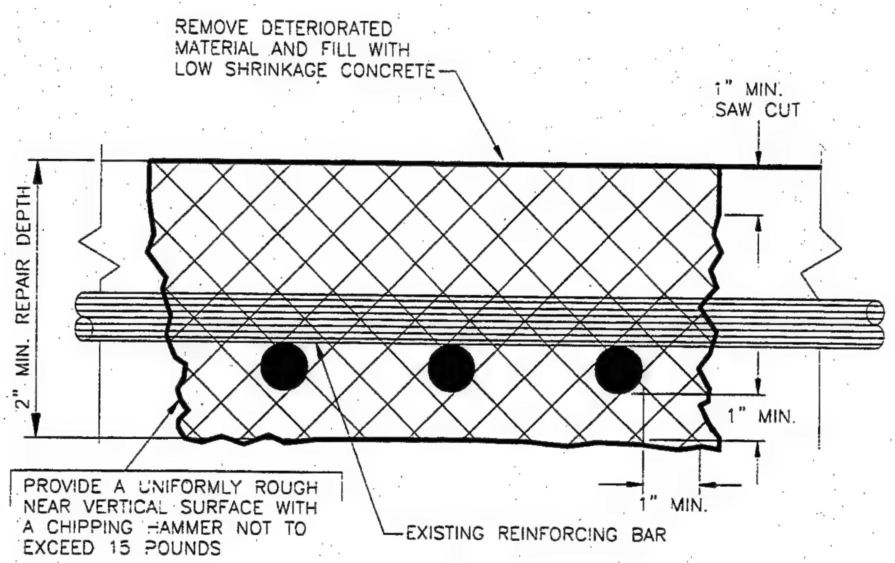
3. LOW SHRINKAGE CONCRETE (OR CEMENTITIOUS PATCH MATERIAL) SHALL HAVE A MAXIMUM ALLOWABLE SHRINKAGE OF 0.05%. (SEE CONTRACT SPECIFICATION FOR MORE REQUIREMENTS.)

4. ALL EXISTING REINFORCING BARS SHALL NOT BE CUT, REMOVED, OR REPLACED WITHOUT THE PRIOR APPROVAL OF THE CONTRACTING OFFICER.

5. USE A REBAR LOCATOR (PACHOMETER) PRIOR TO CUTTING OR REMOVAL OF CONCRETE TO VERIFY DEPTH AND LOCATION OF REBAR. ADJUST DEPTH OF CUT ACCORDINGLY TO AVOID DAMAGING THE REBAR.




TYPICAL REPAIR PERIMETERS

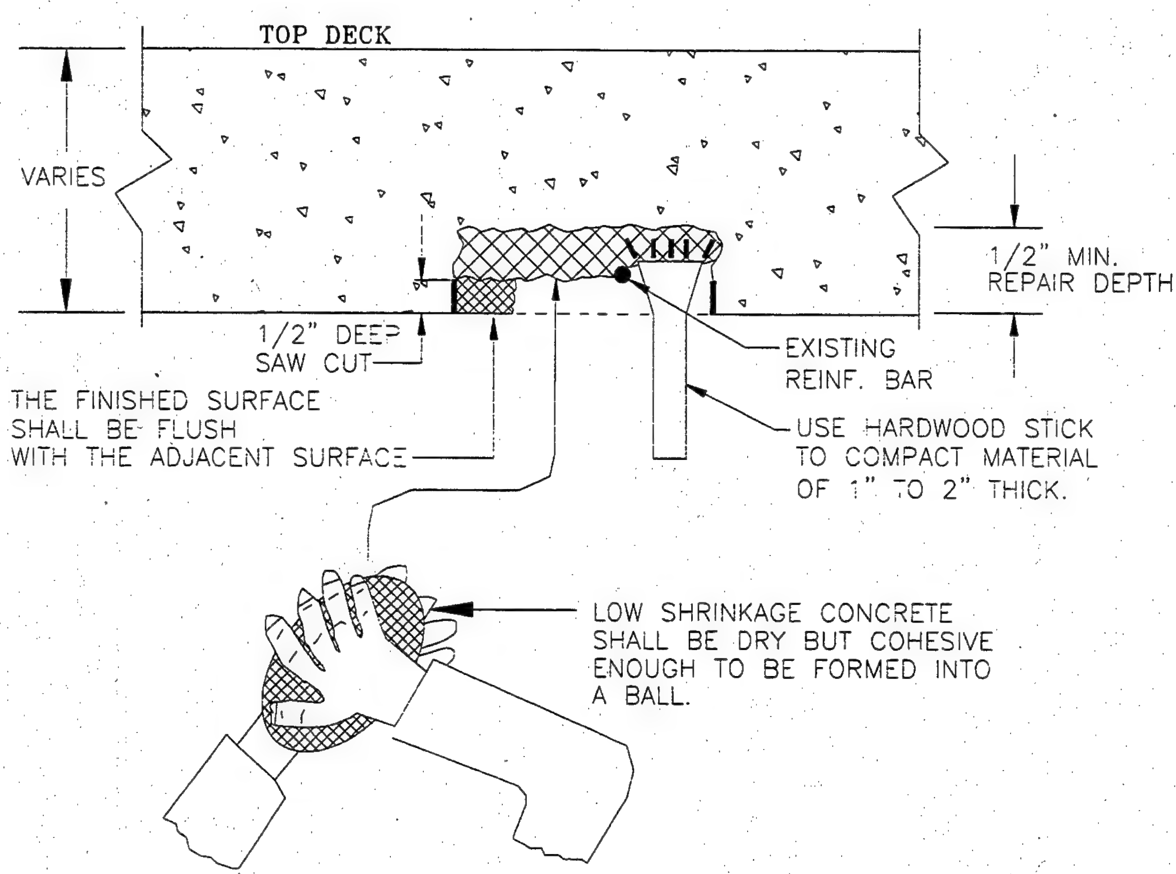



SECTION A–A

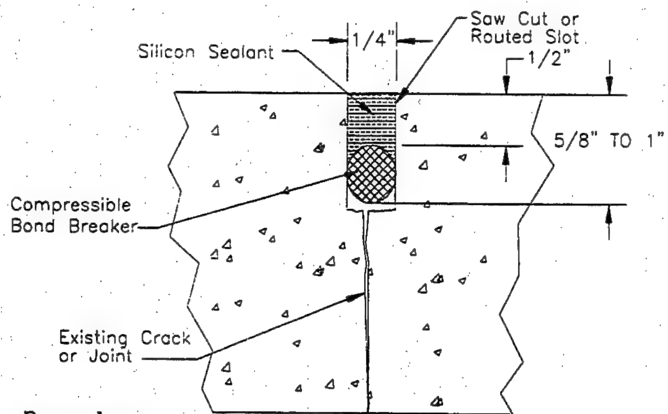
NOTES:

1. SAW CUT (1" DEEP) A REPAIR PERIMETER LAYOUT OF THE DAMAGED AREA.
2. REMOVE DETERIORATED CONCRETE, ABRASIVE BLAST THE STEEL, CLEAN UP ALL CONTAINMENTS & DUST, SATURATE SURFACE WITH WATER FOR 24 HOURS, REMOVE SURFACE WATER, APPLY BONDING AGENT, AND PLACE LOW SHRINKAGE CONCRETE. (SEE CONTRACT SPECIFICATION FOR MORE DETAILS.)
3. CLEAN CORRODED REINFORCING BARS. CUT, REMOVE AND SPLICE WITH NEW REINF. BARS IF THE CROSS SECTION AREA OF THE CLEANED BARS DOES NOT MEET THE MINIMUM REQUIREMENT. SEE CONTRACT SPECIFICATION FOR MORE DETAILS.

## PARTIAL DEPTH REPAIRS



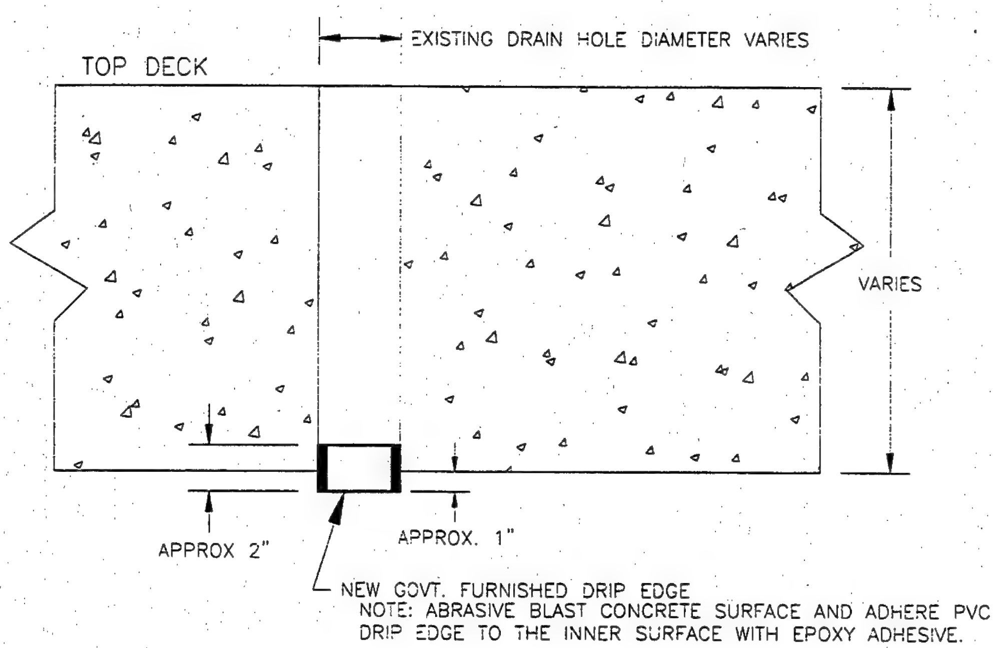

HAND PACK METHOD




## Procedure:

1. The Contracting Officer shall mark with chalk the limits of each crack and joint to be sealed.
2. Straight cracks and joints shall be saw cut 1/4" wide by 1" deep and cracks which are not straight shall be routed 1/4" wide by 1" deep.
3. The edges of the opening to receive the sealant shall be abraded to establish an appropriate anchor profile and then air blasted to remove dust and debris.
4. A primer designed to be used with the sealant shall be applied to the prepared concrete edges prior to placement of the bond breaker in accordance with the manufacture's instructions. The Contractor and the Contracting Officer shall inspect the prepared surfaces prior to placement of the bond breaker.
5. A 3/8" diameter compressible bond breaker shall be pressed into the opening at a uniform depth of 1/2".
6. The sealant shall be placed into the opening in accordance with the manufacture's instructions. Care shall be taken to ensure that no voids exist. Excessive sealant shall be removed from the adjacent concrete deck surfaces.
7. The Contractor and the Officer in Charge of Construction shall inspect and probe the cured sealant to assure that it has adhered to the surfaces.

JOINT AND CRACK REPAIR




DRAIN HOLE DRIP EDGE

# CHAPTER 9

## CASE STUDY: REPAIR OF CORROSION DAMAGED PIER STRUCTURE USING THE FORM AND PRESSURE PUMP METHOD AT PORTSMOUTH, NH NAVAL BASE

# CASE STUDY: REPAIR OF CORROSION DAMAGED PIER STRUCTURE USING THE FORM AND PRESSURE PUMP METHOD AT PORTSMOUTH, NH NAVAL BASE

Prepared for a

**WORKSHOP**

**How to Make Today's Repairs Durable for Tomorrow;**
**Corrosion Protection in Concrete Repair**

March 21, 1998
Houston, Texas

Prepared by
Douglas Burke
Naval Facilities Engineering Service Center

## ABSTRACT

The Navy is continually looking for the best methods to repair its concrete waterfront structures. The goal is to identify methods and materials that provide at least 15 years of service. An evaluation of repairs made to a pier at Portsmouth Navy Shipyard was conducted by the Naval Facilities Engineering Service Center (NFESC). The excellent condition of these repairs after 16 years of service demonstrates that the form and pressure pump method to place a polymer-modified concrete meets the Navy goal. A description of the project, repair methods, and materials is provided.

## INTRODUCTION

Many of the Navy waterfront structures are 25 to 75 years of age and require frequent and expensive repairs. Within a Navy pier there are various types of structural elements such as piles, pile caps, and deck slab, each of which may require repair materials and procedures that are appropriate for its specific function, condition, and accessibility.

The typical Navy marine reinforced concrete structure is contaminated with chlorides and rebar corrosion is ongoing. The concrete is often carbonated, resulting in a weakened cement paste at the surface. Conventional repair methods are specified because complete removal of the chloride contamination or the use of cathodic protection methods are often too expensive and therefore excluded from consideration.

## CASE STUDY

A case study of the performance of concrete repairs, after 16 years using the form and pump method, was performed in September 1996 at the Portsmouth Naval Shipyard in New Hampshire. This was a major concrete repair project in 1980 involving about 24,000 square feet (2,230 square meters) of repair surface. The general contractor was Peabody NE, Inc. The Navy's engineer was C.J. Foster, Inc., and the subcontractor who performed the repairs was Structural Preservation Systems, Inc. The major part of the project included extensive repairs to the pier's concrete support beams and pile caps.

The beams are exposed to severe conditions in a marine environment. They are located in the splash zone, exposed to wetting and drying, freezing and thawing, and elements of sea water (the Portsmouth harbor has 12-foot (3.7-meter) tides). The existing conditions at the time of the repairs manifested in severe concrete spalling, cracking, and corrosion of embedded steel.

This was the first project where the form and pressure pump method was used. Polymer-modified concrete was used as the repair material. In total, 7,000 cubic feet (198 $m^3$) of concrete was placed.

The observations during the case study demonstrate that the repairs are in good condition after 16 years in service. It should be noted that in addition to exposure to the severe environment, the pier structures carry heavy cranes, construction equipment, and heavy vehicular traffic. All these loadings cause vibration and impact in the supporting structures.

Several isolated cases of corrosion of embedded steel and concrete deterioration were found in repairs adjacent to steam pipes. Elevated temperatures associated with the steam pipes accelerated the rebar corrosion and concrete deterioration. In general, one can expect that the intrusion of moisture and corrosive agents will be greater at increased ambient temperature and, hence, the time for rebar corrosion will be shorter.

In conversations with the shipyard engineer, he stated that the repairs were very successful and substantially extended the service life of the repaired piers.

The form and pump pressure method was specified and used by the Portsmouth Naval Shipyard in subsequent concrete repair projects performed in 1983 and 1986.

### Repair Material

Due to the large size of the project, it was determined that the polymer-modified concrete (PMC) was more cost effective than epoxy mortar. Because the polymer modifier replaces an equal amount of water needed for the concrete, shrinkage cracking is reduced, water permeability is reduced, and freeze-thaw resistance increases.

The mix proportions for the project consisted of 3 parts concrete sand, 1 part portland cement, 2 gallons of epoxy, and approximately 3.5 gallons of water.

### Repair Procedure

Deteriorated concrete was removed and the corroded steel was sandblasted to "white" metal.

Form work consisted of 5/8-inch (15.9-mm) and 3/4-inch (19-mm) ply attached to the concrete with expansion anchors. After mixing, the PMC was placed into a positive displacement mortar pump and transported through a 2-inch (51-mm) diameter high pressure hose to the form work. The forms contained a series of ports at 2- to 3-foot (0.6- to 0.9-meter) intervals along the form work. The forms were pressurized to 15 psi.

## DESCRIPTION OF THE FORM AND PRESSURE METHOD

The form and pressure pump method is accomplished by placing the repair material into a closed form with a concrete or grout pump. The forms are usually single face forms, enclosing a cavity in a concrete member, vertical or overhead, such as the side or bottom of a beam, a wall cavity, a column cavity, or the bottom of a slab. The main difference between the form and pump method and conventional pumping concrete in a form is the pressurization of the material mix once the form is full. The special form work design allows for the pressurization to be achieved with the power of the pump. This operation significantly improves the bond between the repair material and the existing concrete substrate, and between the repair material and embedded reinforcement. It also allows for a more complete filling of the repair cavity and better consolidation of the repair material than is generally possible with other repair methods. Combined with the use of relatively low shrinkage repair material, this method allows for durable repairs eliminating or minimizing cracking.

### Forms

In form and pump (F&P) repairs, forms usually are made of wood materials because they are easy to put together and they are lower in cost than other materials.

As with all forms, F&P objectives in form design are:

- *Quality* - to design and build forms accurately so that the desired size, shape, and finish of the repair are attained while placing quality repair material and providing adequate cover over the steel reinforcement.

- *Safety* - to build form work that will safely support all dead and live loads.

- *Economy* - to build the forms efficiently saving time and money. The less pieces, the more it can be reused.

The main difference between F&P and standard forms is that they must be designed to take not only the full liquid load of the repair material, but also the extra pressure when pressurizing. Design of the forms should follow standard practice for cast-in-place concrete construction, except for the form pressure. Forms should be designed to resist a minimum pressure of 15 psi. They need to be made "tighter" than standard forms, especially for overhead placement; watertight if possible. The forms must be held tight against the existing structure to prevent the new material from wedging between the form and the face of the existing structure.

A sealant such as silicone caulking or urethane foam is used between joints in the plywood and sandwiched between the plywood and the perimeter of the repair area to prevent leakage. Penetrations through the form face are also sealed against leakage.

**Placement**

In F&P, the repair material is placed into the forms by a pump. The pump must be compatible with the material being placed, and sized to the quantity of material being installed at any one time.

There are many different types of pumps from large truck-mounted boom pumps of 40 to 60 cycles per hour to small moyno types that are rated in gallons per hour. The pump type required depends on the material being placed more than any other factor. Moyno pumps are for mixes that do not contain coarse aggregate (gravel or stone). They are small, easy to move around, and run on 110-volt electrical supply. They are often used on small projects or where the areas to be repaired are small and require a repair material with fine aggregate only.

Hydraulic swing tube pumps, either truck or trailer mounted, are best suited for repair materials with coarse aggregate. The squeeze type pump can also be used.

After the pump is selected, it is necessary to connect it to the form. This is done with either rubber hose or hose and pipe. Pipe offers less resistance to flow of the material than the hose. However, pipe is not flexible, and is difficult to mount onto the form. If there is a long run (several hundred feet between the pump and the form), a combination of pipe and hose is used. Generally, a section of hose is connected from the pump to the steel line and one or two hose sections are then placed on the end of the pipe line to connect to the forms. If there will be several concrete placements in the same area, the steel line is cleaned in place and left for use on the next placement.

Abrupt changes in line size (from larger to smaller) will cause a blockage most of the time. Long tapered reducers shall be used when changing pipe or hose size. For example, if the line size between the pump and the form is 3 or 4 inches and the pump discharge is 5 inches, use a tapered reducer (5 to 4 inches or 3 to 4 inches) between the pump and the line. At the form end of the line, the line size should be reduced further with a tapered reducer to a 2-inch or 2-1/2-inch hose that will be connected directly to the form valve.

For connections between pipe joints or hose sections, it is recommended that a metal clamp with a rubber gasket be used. It is most important that under pressure these connections don't leak because the mix may become dewatered in the line and cause a blockage. If the connection leaks, it is because of a damaged flange or gasket. Both should be fixed as soon as they are spotted.

It is important to have a gauge in the line where it connects to the form. A recommended gauge size is 200 psi. This gauge capacity is enough to monitor pressure. It is also necessary to have a gauge at the start of the pump discharge line. On hydraulic pumps, an experienced operator can control the line pressure by monitoring the hydraulic system pressure gauge.

Before any pumping starts, a positive, instantaneous communication system between the pump operator and the nozzleman should be established. When the form is full, one stroke of the pump could cause the form to fail if the pump cannot be stopped in time. Do not start pumping until there is direct continuous communication with the pump operator. Two-way radios are best for this purpose.

Precautions should be taken when working with a new pump operator. The operator and nozzleman need to discuss the operation and the signals for directing the pump. The nozzleman should direct the pump operator; the pump operator follows the nozzleman's directions. Only one person on the crew directs the pump, thereby eliminating confusion and possible injury. The pump operator must be instructed to constantly monitor the radio or the head set. The nozzleman may need to stop the pump at any time. The fewer words used in commands to start or stop the pump, the better. They must be clear and concise and said loud enough for the operator to hear. The importance of clear and constant communication between the pump operator and nozzleman cannot be over emphasized in F&P.

**Pump Line Cleaning**

After completion of pumping, the line and valves must be cleaned. Be sure to have a proper line sized "go devil" on hand to clean out the line. After the pump pulls as much material from the line as it can, by running in reverse, the "go devil" is inserted in the form end of the line, and an air line attached behind the "go devil." The remaining concrete is "blown" back to the pump end. Care must be taken to secure the pump line and catch the material including the "go devil" exiting from the line. This procedure can be reversed if it is easier to handle the waste concrete in the line at the form end of the operation.

**Pumping**

If material is being delivered from a ready mix plant, after the truck is "mixed up" and ready to start discharging, a small amount of the concrete should be discharged into a wheelbarrow to check for proper mixing and slump. Do not put material into the pump hopper before checking it. If cylinders and/or sump tests are to be taken, the material in the wheelbarrow can be used in order to not hold up the placement operation. If the concrete does not appear properly mixed or has an inadequate slump, don't use it. The cost to remove the material, once it reaches the form, is very high.

After the quality check on the material, slick line (cement mixed with water) is added to the pump hopper and started through the line. This material will "wet out" the inside of the

pump line, help seal joints, and prevent the concrete from blocking up in the line. Slick line consists of one or two bags of cement with approximately 5 gallons of water per bag. It is mixed in a mortar mixer, or in buckets, and dumped into the pump hopper. After most of the slick line has been pumped from the hopper through the line, the mixed repair material is placed in the hopper and pumped through the line, behind the slick line.

On the form end of the line, the crew is standing by with a wheelbarrow, buckets, or a 55-gallon drum to catch the slick line material. Do not pump the slick line into the forms, waste it. When the repair material starts to flow from the hose, stop the pump and clean off the connection at the end of the hose line, and connect it to the form.

There should be one or more pumping ports already attached to the form. The hardware should be in place with valves ready for the hose attachment. Start at the bottom or far end of the form, pumping from bottom to top, or from one side to the other, and start with all valves open. As material starts to exit from the valves and/or vents, close the valves and plug the vents. Forms may be pumped from a single port. This is possible if the pressure remains low and the concrete is traveling to all areas of the form. If the pressure starts to rise significantly, the valve shall be closed and the hose disconnected from that pumping port and reconnected to another port where material has already exited. Avoid trapping air between two pumping ports by skipping around. The nozzleman and helpers should have a bucket of water ready to clean off the connections as they change ports. It is necessary to wash the concrete out of the rubber seal and the clamp and grooves in order to reconnect the hose to the pumping ports.

**Vibration**

For sections 3 to 6 inches deep, vibrate the exterior of the form as the concrete is being pumped in. On deeper placements, as required for Pier 12, it is necessary to install access holes in the form to insert the vibrator. These must be plugged before starting to pressurize the form. Vibration after pressurization may cause unwanted movement or may overstress the forms. Do not vibrate after the forms have been pressurized.

**Pressurizing**

Pumping will continue until the form is full. At this point, the nozzleman requests the pump operator to give short strokes and monitors the form carefully. He is watching and listening for “cracking” in the form indicating that pressure is straining and/or slightly bulging the form, also an indication that pressure is building inside the form. Once this condition has been achieved, all vents should be capped or plugged off, and all valves closed. The hose can be disconnected from the form.

After the repair material has reached a stage where it will not flow out of the valve (about 20 minutes to 1 hour), unscrew the valves and nipples from the flanges and clean them using a water hose. In most cases the concrete is still green, and a screwdriver and water will remove the material. A wire brush is also helpful to clean the threads. Most of these valves can be taken apart and the ball cleaned and regreased. It is important to take the valves apart and regrease them because the fines will build up and prevent the ball from operating. The valve will now be ready for reuse.

**Curing**

In the F&P method, the forms must stay in place until the repair gains design strength and becomes self supporting. If the forms are removed too soon, the new material may sag, break bond, and crack. In hot, dry environments, it may be desirable to wet the forms during the curing process to minimize water loss from the concrete and to keep the temperature down. After the forms are removed, the repair should be wet down several times. As soon as the surface becomes dry, immediately apply a curing compound.

## CONCLUSIONS

The form and pump method combined with good quality control practices is a good method to repair deteriorated concrete waterfront structures. The expected performance life can exceed 16 years where the ambient temperature is similar to New Hampshire. Performance life for repairs made in warmer climates will be shorter.

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# CHAPTER 10

# CASE STUDY OF DRYDOCK NO. 6

## BACKGROUND

Drydock No. 6 is more than 35 years old. It originally cost about $175 million and the estimated replacement cost would exceed $500 million. Currently, repairs to the concrete are necessary because of corrosion of the steel reinforcement. In 1992, Puget Sound Naval Shipyard (PSNSY) contracted with BERGER/ABAM Engineering, Inc., to prepare construction documents to address the repair of cracks, spalls, and delaminations (Refs 1 through 4). These documents address other repair issues that are not part of the scope of this report. BERGER/ABAM hired Construction Technology Laboratories to conduct a site specific condition survey of the concrete (Ref 5). The shipyard did not award the repair contract as planned, probably due to funding limitations. The original documents are now being updated and revised by BERGER/ABAM Engineers in accordance with Reference 5. PSNSY has requested that NFESC review the project documentation, visit the site, and prepare written recommendations according to a statement of work (Ref 6). BERGER/ABAM is currently updating the condition survey by sounding the walls to the height of 8 feet to obtain a representative sampling of how much the deterioration has progressed in 4 years.

## REPAIR OBJECTIVES

Due to the high replacement costs, lack of MCON funding for new construction, and continued mission requirements, PSNSY's objective is to ***extend the performance life of Drydock No. 6 for as long as possible.*** The objectives of the repairs are to:

- Minimize the frequency of falling pieces of concrete.
- Provide long lasting repairs.
- Improve the appearance of the walls.

The scope of work refers to "structural repairs." Typically, it is very difficult to execute the repair in a manner that provides for the distribution of structural loads through the patched concrete. A structural analysis would likely show that the loss of 6 inches on the surface of the thick cantilever walls is not significant to the performance of the structure.

## CORROSION MITIGATION OBJECTIVES

The corrosion mitigation objectives are to:

1. Fill all cracks to minimize penetration of corrosive species to the depth of the steel reinforcement.

2. Apply a penetrating surface sealer to the entire wall area to forestall, retard, or stop any ongoing corrosion processes.

## SITE VISIT

The site was visited 6 and 7 January 1997. Joe Sullivan (Ref 7) was the point of contact from the shipyard and Joe Stockwell represented BERGER/ABAM Engineering. The drydock was visually examined and random soundings were made. The Public Works Commanding Officer and Executive Officer were briefed prior to departure. Reportedly, this is the first major repair effort.

The mechanism for deterioration is rebar corrosion due to ingress of salt water to the depth of the steel, especially through vertical control joints and drying shrinkage cracks. About 80 to 90 percent of the wall area appears to be in very good condition. Three large delaminations were indenified as needing to be knocked down immediately to minimize the hazard of falling debris. Standing water was observed in the service gallery. It appears that this water is contributing to the moisture in the vertical cracks and subsequent corrosion of the steel reinforcement. It is recommended that corrective measures be taken.

Prior repairs performed on Drydock No. 3 using pressure injected epoxy and polyurethane grouts were inspected. Both appear to be in excellent condition after 1 to 2 years of service. Concrete repairs using form and pump techniques and machine-applied methods also have performed very well.

## HIGH PERFORMANCE REPAIR MATERIALS

### Compressive Strength

Compressive strength is typically the defining criterion for the selection of concrete repair materials. In this application, the concrete patch will probably never carry any significant compressive loads. Although cementitious materials have excellent compressive strength characteristics, performance in this case is not a function of compressive strength and need not be a criterion for material selection.

### Tensile Strain Capacity

The repair material is subject to drying shrinkage, temperature changes, and flooding the drydock; these will all induce tensile strain. To avoid cracking and delamination of the concrete repair material, one needs to specify a repair in terms of *tensile strain capacity*. Unfortunately, the industry has not yet devised a test which can be correlated to this characteristic. It is known that tensile strain capacity is related, in part, to drying shrinkage, creep, and the modulus of elasticity. A maximum allowable shrinkage and an appropriate test method is recommended.

Some cracking of the repair material will mostly likely occur. Our objective is to minimize the size and frequency of the cracks by using reasonable specifications and qualified inspection to obtain good workmanship and high quality.

## STRESSES

Many different loads affect the stresses in the repair material. All materials change volume when subject to stress. The combined stresses on the repair can result in cracking. Cracks relieve the stress but also allow the direct ingress of corrosive species. Cyclic stresses beyond the capacity of the material can result in progress cracking, delamination, and spalling. The combined tensile stresses resulting from drying shrinkage, temperature changes, and flooding need to be resisted by the repair materials.

### Drying Shrinkage

Drying shrinkage is a one time event related to the composition of the material, the placement, and curing conditions. As the material cures it shrinks. Because the repair is bonded to the substrate, it is restrained and hence the shrinkage results in tensile stresses in the repair. By curing the repair properly, one can minimize cracks that result from shrinkage stresses.

### Temperature Changes

Temperature changes produce cyclic stresses. As these tensional loads are applied to the repaired area, the combined internal tensile stresses may exceed the tensile strength capacity of the repair material.

### Operational Flooding

Operational flooding produces cyclic stresses. The concrete repairs are made when the surfaces of the walls are in compression. When the drydock is flooded, the hydrostatic forces will push against the cantilever walls, tending to elongate the exposed surface of the walls. The compressive forces in the *unrepaired wall* will be reduced or reversed. The patched areas will also be elongated, thus inducing tensile stresses in the repair material. These tensile stresses are additive to other stresses.

## CRACK INJECTION

Cracks provide direct access for corrosive species to attack the steel reinforcement. Epoxy crack injection can be an effective way to seal stationary cracks. Since epoxy is rigid, it is not effective in sealing cracks that are subject to movement. Cracks move for a number of reasons, such as thermal expansion and contraction, loading, and corrosion of the rebar. It is a common mistake to use epoxy to repair cracks that are caused by rebar corrosion. The expansive

steel by-products result in growth of the crack. Epoxy is not sufficiently flexible to accommodate these movements without failure at or near the interface, therefore failures often occur within 1 or 2 years. Most cracks in the drydock are moving or have the potential to move, therefore it is recommended that a flexible sealant be used for the repair of all cracks.

## PENETRATING SEALERS

All of the walls of the drydock are to be sealed after the repairs are made, this is intended to slow further chloride penetration and further rebar corrosion. The use of sealers over a new cementitious patch is an effective primary method to retard or to restrict the ingress of corrosive species into the concrete. Sealers are not permanent, and periodic reapplications are necessary. Sealers typically penetrate only a few millimeters. The 1992 condition assessment contains data from several core samples, all of which were taken in cracked or delaminated concrete, except for Core No. 11, which was taken in sound concrete. At this location, chloride ion contamination at 3 inches deep was 1.9 pounds per cubic yard (Ref 5). In the presence of moisture and oxygen, *which is sufficiently available*, the steel rebar is probably corroding. The application of a penetrating surface sealer at the Core No.11 location will slow the ingress of chloride penetration. Given the age and condition of the structure, assuredly, additional areas are contaminated and are experiencing ongoing rebar corrosion, especially along the joints and cracks. Eighty to ninety percent of the surface area appears to be in very good condition and it is recommended that a penetrating sealer be applied over the entire wall to retard the ingress of chlorides.

The frequency of reapplication of the penetrating sealer is uncertain. It is recommended that the chloride ion contamination be measured annually at three specific locations at 1 inch, 2 inches, and 3 inches deep so as to monitor and document the level of contamination. By doing so, the command will have data to help make decisions on the frequency of reapplication and its effectiveness.

## PERFORMANCE EXPECTATIONS

The performance life of a repair is extremely difficult to predict, as the performance life depends on the material selection, method of application, and quality of the workmanship. The ROICC should strive to obtain the highest quality workmanship from the contractor. Continuous inspection of the repair process is a method to help maintain uniform and continuous high standards throughout the project. The need for ***continous qualified inspection*** of the repair contract can not be over emphasized. If all of these items are correctly accomplished, the repaired areas should function for 20 years.

Outside the repair areas the interior and external environmental conditions can produce new delaminations. It is common to see new delaminations occur adjacent to patches in 1 to 3 years. There are two dominate reasons why this occurs. They are:

- The extent of deterioration is often underestimated. Because the *area must be chipped back to sound concrete and to uncorroded rebar,* it is impossible to foresee the repair boundaries in a non-destructive condition assessment. Good inspection can minimize this problem.

- Rebar corrosion is accelerated by differences in conditions along a continuous rebar. After the high quality repairs are completed, a portion of the rebar is protected by the new high quality patch, while the steel extending into the adjacent concrete is exposed to relatively severe conditions. These differences promote a strong corrosion cell resulting in accelerated rebar corrosion and possible delamination of the concrete near the patch and occasionally at the patch itself. Life extension methods can address this problem.

## LIFE EXTENSION METHODS

The use of impressed current cathodic protection (ICCP) can be used to arrest the corrosion process. This method can provide a 15- to 20-year solution. However, there are many issues that must be addressed and considered in the design, installation, and operation of an ICCP system. The application of the ICCP system requires several protection zones, each with electrical supply and control circuits. The external application of a flame sprayed titanium distribution anode may not be sufficiently durable to resist the wear and tear on the lower portion of the wall. It is possible to cover the anode with epoxy or to cut grooves and embed the anode.

In 1993, NFESC prepared a comprehensive technical assessment of the technology (Ref 8). Currently, NFESC is performing cooperative research with industry to resolve installation and performance issues. In FY97, NFESC plans to install a 1,000-square-foot demonstration of an ICCP system on the substructure of a Navy pier at SUBASE San Diego. The system performance will be monitored for at least 1 year. Results after 1 year of operation will provide valuable installation and performance data necessary for the transition of this technology to Navy marine structures. It is recommended that the use of this technology for Drydock No. 6 be postponed until these tests are completed.

Chloride ion removal techniques may be a viable candidate for future consideration. An assessment of the feasibility of employing this emerging technology is worthy of consideration. NFESC will attempt to include this investigation into the current scope of work funded by Naval Facilities Engineering Command Headquarters (NAVFACHQ).

## ECONOMY OF ICCP AND CHLORIDE REMOVAL

Currently, PSNSY needs to spend at least \$6 million to make repairs. With a rough estimate at \$20 per square foot, \$3 million would be required to install an ICCP system to stop the corrosion cell. The life expectancy of this system is 20 years and would eliminate the need to spend \$6 million every 5 to 10 years to repair the concrete due to rebar corrosion. Electrical continuity must be determined. Typically, structures contain sufficient continuity, but one must

confirm it. If technically feasible, the benefits-to-cost ratio of chloride ion removal will probably be comparable to using ICCP.

## REPAIR TECHNIQUES AND MATERIALS

The recommended repair material and application method is based upon the following criteria (this list was developed from experience on other Navy jobs and discussions with industry and Navy experts):

- The operational costs associated with repairing a drydock dictate that, within reason, the very best repair technique and materials should be selected.
- The ultimate long-term properties of the repair material are far more important than the ease of application.
- Bond of the repair material to the concrete substrate depends mostly on mechanical interlocking.
- There are no repair techniques or materials that are tolerant to the applicator's lack of experience, workmanship, and quality control.
- Segregation of the repair material will alter the repair material's physical properties.
- The greatest problem in concrete repair is to minimize the amount of cracking. A network of micro-cracks and visible cracks provides transport for corrosive agents to the rebar.
- When selecting the repair method, it is best to specify only one and at most two methods for the same project.
- The use of *machine applied material (shotcrete*) is especially sensitive to applicator experience, materials and equipment, workmanship, and quality control. Generally, its use is discouraged. Improper surface preparation is largely responsible for failure of the repair. Large cumulative areas of repairs, less than 3 inches thick and containing very little rebar, may be most efficiently repaired using shotcrete, although *hand application* methods typically perform better.
- Vertical repairs containing closely spaced rebar and more than 20 square feet and at least 3 inches deep are best performed using the *form and pump* method.
- Hand-applied dry packing is best for small areas.

## REPAIR MATERIALS

It is recommended that a prepackaged repair material be used to achieve the low shrinkage required. Shrinkage is the most important property. Because of product variability it is recommended that a third party independent testing laboratory verify that the material delivered to the job site meets the criteria.

Drying shrinkage shall not exceed 0.05 percent at 28 days per ASTM C 157 modified to use molds per ASTM C490 (3 x 3 x 11.25 inches) with a 10-inch gauge length. During the first 24 hours of curing, the molded specimen shall be cured at 46 to 54 percent relative humidity at 70 to 76°F. After 24 hours, remove the mold and cure as prescribed in the standard.

The following prepackaged products have been used and tested for other repair projects and have properties consistent with the objectives of this project:

| Manufacturer | Phone | Product Name |
|---|---|---|
| Five Star | 202-336-7900 | Five Star Structural Concrete V/O |
| Master Builders | 216-831-5500 | Emaco S66-CR |
| Fosroc | 800-441-3633 | Renderoc LA |
| Sika | 800-933-7452 | Sika Top 111 Plus |
| Euclid | 216-531-9222 | Euco SR-93 |

## DRY PACKING AND HAND-APPLIED METHODS

Dry packing is the recommended technique to repair small cavities. Hand placement can also be an acceptable method. Both application methods are similar and the contractor should follow the manufacturer's recommendations. In both methods, it is very important to have good consolidation in and around the rebar and between the layers. A vertical shoulder is necessary to provide lateral restraint when packing or placing the material. A shoulder will be established at the perimeter by cutting the concrete with a saw and chipping the interior. The material shall meet the shrinkage criteria of 0.05 percent.

Dry packing is a repair method of placing zero-slump, or near zero-slump, concrete or mortar by ramming it into surface cavities. Sufficient water should be used to produce a mix that will stick together while being molded into a ball with the hands and that will not exude water but will leave the hands damp. Less water will not make a sound, solid pack, and may result in excessive shrinkage and failure.

Hand-applied techniques use a non-sag material often with special blends of cement. Skill of the applicator is important to obtain good consolidation and bond to the substrate.

A bonding agent consisting of either neat cement, cement-sand, or latex-cement-sand slurry shall be used because dry pack lacks the extra moisture necessary to promote good bond. Compaction densifies the repair material and provides the necessary contact with the existing concrete for achieving adequate bond. The action of the trowel is used in hand placement to accomplish sound consolidation. In the dry pack method, a hardwood stick and a hammer is used. These sticks are usually 8 to 12 inches long, and are used in preference to metal bars

because the latter tend to polish the surface of each layer and thus make bond less certain and repair less uniform. Much of the tamping should be directed at a slight angle and toward the sides of the cavity to assure maximum compaction in these areas. Dry pack and hand placement are usually placed in layers depending on the repair thickness.

Because of the relatively small volume of most repairs and the tendency of old concrete to absorb moisture from new material, water curing is necessary for the first 72 hours followed immediately by a sprayed curing compound.

## FORM AND PUMP

The form and pump method is the recommended method to place concrete repair material. The conventional application of this method has a shortcoming in that the repair material typically does not completely fill to the top of the form, leaving a small gap at the top. This crack must be filled. Options are to inject the crack or to hand pack a cementitious mortar into a routed out joint. Alternatively, the repair material can be placed using the pressure pump method, which fills the form more completely. Another variation is to prepack the form with a graded aggregate and then pump a very high slump cementitious slurry. NFESC recommended this technique in June 1987 at Camp Courtney, Okinawa, Japan with success (Ref 9).

## FORM AND PRESSURE PUMP

The form and pressure pump method was first used by the Navy at Portsmouth Naval Shipyard in 1983 and 1986. A site inspection performed in September 1996 showed these repairs to be in excellent condition. The repair material is placed into a closed form with a concrete or grout pump. The forms are usually single faced and pressurization is achieved with the power of the pump. This operation significantly improves the bond between the repair and the substrate and between the repair material and the embedded steel reinforcement. It also allows for a more complete filling of the repair cavity and better consolidation than conventional methods. ***Use of a very low shrinkage repair material with the form and pressure pump application method is considered the best alternative and therefore it is the recommended method.***

## SURFACE PREPARATION AND CONDITIONING

Surface preparation and conditioning is a critical phase of the repair process. ***Continuous and knowledgeable inspection*** is recommended to assure high quality surface preparation.

The limits of the repair areas should be marked and a decision made on how to "square up" or combine the adjacent areas to simplify the repair geometry and reduce boundary edge length. Excessive or complex edge conditions result in shrinkage stress concentrations and cracking

Most of the removal work is done by small hand-held chipping hammers because of the mobility and versatility these tools allow. In addition, they do the least amount of damage to the remaining concrete and reinforcement. Impact hammers in the 15-pound class are recommended. Impact tools greater than 15-pound hammers can cause cracks in the sound concrete and should be strictly prohibited. The chipping hammer provides a very rough surface texture which improves aggregate interlock at the bonding surface.

***The inspector and contractor should be instructed*** to start chipping a few inches away from the boundary lines. After the rebar is exposed it may be necessary to modify the boundary layout. Rebar with visible corrosion must be "chased" with the chipping hammer to the point where no surface corrosion is visible. Care must be taken to not damage the rebar. It then may be necessary to redefine the boundary of the repair area. Saw cut the perimeter at 90 degrees at least 1 inch deep. The depth of removal is determined by the depth of unsound concrete, but must be at least 1 inch behind the rebar. When completed, the surface shall be clean, sound, and with uniform roughness. After the forms are constructed and prior to placement, the surface should be "Saturated Surface Dry" (SSD). Never pump repair material into a form containing standing water.

## FORMS

Forms are typically custom made of wood. They must be designed to resist the full liquid load of the repair material plus 15 psi from pressurization of the repair when using the *form and pressure pump method.* The forms need to be secured tightly to the wall to prevent the repair material from wedging between the form and the face of the wall. Expansion anchors are typically used. The boundary needs to be sealed with urethane or silicone caulking to prevent leakage of the liquid cement mortar. Excessive loss of liquid cement mortar results in honeycombing of the repair material, allowing rapid ingress of salt water to the rebar.

## PLACEMENT

Placement of the material is by pump. The pump must be sized to be compatible with the materials and size of the repair. Many types of pumps are available. For repair sections greater than 3 inches thick, vibrate the exterior of the form as the concrete is being pumped. Pumping will continue until the form is full. A pressure gauge installed on the supply hose near the form is monitored carefully. Care should be exercised to observe and listen to the forms for distinctive bulging and cracking. Vents are capped off and the supply valve is closed. Do not vibrate the forms after pressurization. Remove the supply hose. Allow the material to take an initial set, about 30 to 60 minutes, then remove the valves and nipples for cleanup. Leave the forms in place for as long as practical, a minimum of 48 hours. Immediately apply a curing compound to the surface.

**SITE VISIT - May 1998 Concrete Repairs Delivery Orders 007 and 0083**

**Objective**

The objective of this section is to document the findings and recommendations of the site visit performed on 21 May 1998 at Puget Sound Naval Shipyard (PSNSY), Bremerton Washington, Drydock No. 6.

**Scope**

The scope of this effort includes:

1. Conduct a site inspection.
2. Recommend testing to access the effectiveness of the concrete repairs.
3. Recommend any rework.
4. Identify any "lessons learned" to be incorporated into the remaining work.
5. Provide an outbrief to PSNSY personnel.
6. Provide a written summary of findings and recommendations.

**Contract Documents**

The project specifications are the link between the owner's vision and the construction of the project. They provide a written vehicle between the Navy and the contractor to meet the Navy's needs. To accomplish this goal, the specifications must be easy to understand and to implement, therefore, good specifications are fundamental to the projec's success. Oversimplification or ambiguity of specifications can lead to confusion, overbidding by the contractor, and poor quality.

The repair strategy was to use a method known as form and pressure pump. This method was pioneered for the Navy at Portsmouth Naval Shipyard in 1980. The Naval Facilities Engineering Service Center (NFESC) inspected the form and pressure pump repairs at Portsmouth in 1996 and found them to be in excellent condition.

Crack repair procedures were modeled from successful crack injection used on Drydock No. 3 PSNSY in 1994. These repairs have performed very well.

The contract documents are complete and clearly written. They delineate state-of-the-art concrete repair procedures using conventional methods and materials. The repair methods specified have a track history of being constructable and durable.

**Site Inspection**

Access to the wall repairs was limited to the areas from the drydock floor to an elevation of about +7 feet. In addition, the repairs in the galleries were inspected. The repairs include four types:

1. Wall repairs using the form and pressure pump method with Renderoc LA.
2. Surface repairs using the hand-applied method with Fosroc SP25 for small areas.

3. Crack injection using high pressure and mechanical packers using WEBEC 1403.
4. Application of a penetrating sealer.

**Form and Pressure Pump Wall Repairs**

The material used for the wall repair was Fosroc Renderoc LA. No data was found in the contract files to document that this material passed the required tests for shrinkage per American Society for Testing and Materials (ASTM) A157 modified. Independent tests by NFESC, not related to this investigation, indicated that this material has a very low shrinkage rate of 0.02 percent, which is less than the criteria of 0.05 percent. The surfaces are cracked more than expected.

There are several reasons why cracks occur in a concrete repair including: plastic shrinkage, drying shrinkage, reflective cracking, changes in temperature and humidity, disbondment, incorrect placement, improper joints, and inadequate curing. Although the cracks in these repairs are more frequent than expected by the Navy, they are within the allowable tolerances for shrinkage and are consistent with typical construction practices. Moisture was apparent at the surface of many of the cracks and that is an indicator that the repairs may not last as long as the Navy expected.

Generally, the form and pump repairs appear to be sound, they "ring" when tapped with a hammer. Most of the repairs have a random-shaped perimeter. Core locations were identified to obtain samples to evaluate the bond and consolidation of the repair material.

Core R1 was taken from the northeast wall, station 15+00E elevation 80 feet. This core, like the others taken for this investigation, was 4 inches in diameter and about 10 inches long. The repair material in core R1 was about 6 inches deep. Visual examination of core R1 shows a sound well-consolidated repair that is securely bonded to the concrete substrate. Core R2 was taken from the northeast wall, station 15+10E elevation 97 feet. Core R2 was well consolidated and broke in half near the interface between the repair and the substrate and the bond appears adequate. Core R3 was drilled from the northeast wall, station 14 +40 elevation 77 feet and was well consolidated and bonded to the substrate. In addition, the core broke in half about 3 inches deeper than the repair interface. At this location, a large lens of sand and water was present.

Most of the form and pump repairs appear to have <u>not</u> been filled correctly. Reportedly, the forms were not sufficiently strong and consequently the forms bulged and in some cases blew out. Evidence of cement adhered to the wall below the repairs seems to confirm this. The projected surface of the repair material from the original surface profile is acceptable from a durability and performance point of view. The abrupt edges were ground to transition with the adjacent surfaces. These cut surfaces contained very high amounts of entrapped air which is likely to be very permeable to the ingress of chlorides, water, oxygen, and carbon dioxide; all of which will shorten the life of the repair.

The technique used by the contractor to prepare the perimeter was inconsistent. The drawings require a 1-inch vertical shoulder at the perimeter. It appears that the perimeter was not always cut at 90 degrees to the surface but was chamfered at up to 45 degrees and filled with a fillet of concrete. Feather repairs and edges cut at 45 degrees are susceptible to spalling and several repairs in the southwest corner at about station 5 have already disbonded.

The forms were not filled completely and often a void was left to be filled in multiple lifts. Reportedly, the forms were not consistently vented which prevented the material from

filling the forms completely. The void at the top of the form was filled by hand placement with Fosroc SP25. This should not have occurred and, as a result, vastly increased the quantities of hand-placed materials. Generally, materials placed by hand will be of lesser quality than materials placed by the form and pressure pump method. Therefore, this defect will manifest as poorer durability for the Navy.

**Small Area Repairs by the Hand-Applied Method**

The hand-applied method was specified for areas less than 2 square feet. The material selected for these repairs was Fosroc SP25. In addition, it was used for the areas not properly filled by the form and pump method. No documentation of the shrinkage characteristics of Fosroc SP25 was found in the contract file. Many of the repairs performed with Fosroc SP25 were tapped with a hammer and sounded "dull." Their integrity and bond to the substrate of all of the SP25 repairs are questionable. Fosroc was contacted to inquire about the shrinkage properties of Fosroc SP25 and they reported that the product had been discontinued. One should question if the product used on Drydock No. 6 was fresh and within acceptable shelf life. Shrinkage data from Fosroc, using ASTM 157, unmodified, indicates that the material will likely not conform to the job specifications. Core R4 was taken through the SP25 repair. The thickness of the repair in this core was about 4 inches. The SP25 was completely saturated with water and unbonded to the substrate. The repair material acts as if it were a sponge, soaking up water. In addition, the core contained a small bolt of unknown origin. It is recommended that all repairs done with this product be removed and reworked correctly.

**Crack Injection**

Procedures for proper crack injection require that cracks greater than 0.020 inches wide must be sealed with an epoxy crack sealer (03931 section 3.1.1 of the specifications) and smaller cracks may also require sealing. "Before a crack can be injected, it must be sealed at the surface *(with a cap)* to prevent the resin from escaping. No problem has frustrated the crack injection process as much as cap leaking. If the cap is not installed properly, the consequences are costly. For the cap to bond properly to the concrete, the surface must be sound, clean and dry. The cap must be rigid to keep injection pressures from causing it to quickly blister and rupture or slowly peel away. A thick cap, 1/8-inch minimum, 3/16-inch optimum, will stay rigid. A high-modulus, 100% solids, moisture-tolerant epoxy is often the resin of choice for capping. ASTM C 881, Types I and IV, are usually most appropriate." [1]

- Core C1 contained resin in a delamination. The resin did not appear to be bonded adequately to the concrete. An injection porthole of about 1/2-inch diameter was intersected by the core, it should have been filled with urethane or cementitious material but was not.
- Core C2 contained an injection porthole that was not filled with resin or concrete. The core also contained a delamination that had some resin in it that appeared not to be adequately bonded to the concrete.

[1] Trout, John, Epoxy Injection in Construction, The Aberdeen Group 1997

- Core C3A was taken through a crack with a cap. The cap was very thin, about 0.01-inch thick, and therefore not sufficiently rigid. The crack had no resin in it.
- Core 5 was taken through a crack which was about 0.02-inch wide. There was no cap over the crack. The crack was not completely filled with resin and the bond of the resin to the concrete appeared inadequate. There was a delamination at a depth of about 4-inches and there was some resin in the delamination which was not well adhered.

In summary, pressure crack injection of the walls appeared to be grossly inadequate. Surface preparation prior to application of the cap appeared to be entirely inadequate. None of the cracks appeared to have been sealed correctly with a rigid cap. None of the injected cracks inspected by core examinations were satisfactorily repaired. It is recommended that the injected cracks be completely (100%) reworked, unless that contractor can demonstrate on a case-by-case examination that the crack has been repaired correctly.

**Penetrating Surface Sealer**

For penetrating surface sealers to seal the wall from the intrusion of salt water, they must be applied to a clean surface. This essential aspect of the work was clearly stated in 07180 section 3.1.2). No attempt was made by the contractor to clean the surface prior to application of the sealer. The contractor should have scraped all of the cement slurry, cleaned off contaminates, and water blasted. In addition, no documentation was discovered in the contract file to verify that the quality assurance requirements stated in 07189 section 1.8 were accomplished. No tests were performed to detect the presence of the sealer. The effectiveness of the sealer as applied, if applied, is probably nearly worthless because the surface was not properly prepared. This work should be redone properly.

**Recommendations for Rework**

**Wall Repairs.** Identify edges that are likely to disbond and repair them according to the specifications with a 1-inch vertical shoulder. Never permit fillet repairs and featheredges. Remove all repairs done with Fosroc SP25 and rework correctly.

**Cracks.** Rework all (100%) of the cracks to assure that the cracks are completely sealed.

**Surface Sealer.** Remove debris, clean and apply the sealer properly.

**Recommendations for Remaining Repairs**

**Preparation.** A vertical shoulder at the perimeter of the repair is necessary to prevent spalling of the repair at the edges. This practice was not always followed, it is important to do so in all future work to avoid spalling at the boundary.

**Placement.** Good construction practices for using the form and pressure pump method are contained in NFESC "Concrete Repair Recommendations and Specifications," by Douglas F.

Burke, April 1997. Future work should follow these guidelines. A minimum number of cold joints between vertical lifts are desirable, 10-foot lifts should be a minimum goal. The forms must be filled completely and under 15-psi pressure to assure maximum bond and durability.

**Cracks.** Existing cracks that coincide with wall repairs will reflect through the bonded repair. These cracks must be injected prior to the concrete repairs.

**Quality Control.** The literature recommends that strict adherence to quality control is necessary to assure that good concrete placement practices are followed. The Job Order Contract must contain necessary quality control requirements to assure that the repairs are accomplished satisfactorily. Work to date indicates a lack of attention to quality assurance issues related to long performance and durability of the repairs. A review of these procedures is recommended. A meeting is recommended to discuss the procedures prior to continuation of the repair work. Prior to continuation of the work on the head wall, the contractor shall successfully demonstrate to the Navy a successful method of forming, placing, and curing Renderoc LA using the pressure pump method. In addition, methods for crack injection should also be demonstrated. Continuation of the project should not proceed until all parties have witnessed and approved the procedures. Completion of the demonstration repair will set the standard for all future work.

**Sealers.** Penetrating sealers must be applied after the surface is properly cleaned and the technical representative has trained the contractor.

**Commentary**

**Repairs.** About half of all concrete repairs fail in the first few years and 99 percent of those fail due to either the use of inappropriate repair materials or inadequate surface preparation, or both. Cracks and disbondment are usually progressive and will continue until the function of the facility becomes impaired or unsuitable for its intended purpose. The hydrostatic head behind the walls will force water through any cracks not repaired in the wall. Water is very detrimental to concrete durability.

**Crack Injection.** Injection of the drydock is likely to require extremely high quantities (cubic feet) of resin to fill the cracks and the myriad of unknown intersecting voids in the concrete wall. When the surface of the crack is sealed, the resin is forced into the cracks and intersecting voids until all of the spaces are completely filled and enough back pressure develops to force the material out of the next injection port. If the crack goes completely through the wall, then the resin may flow into the backfill. In the galleries both sides of the crack must be sealed prior to injection. The contractor should add an accelerator to the resin so that it sets prior to flowing out the backside. The amount of pressure used during injection is also critical and directly affects the amount of resin that will be used. This aspect of the project is most difficult to administer because the contractor is being paid by the linear footage and not by total volume of resin used. Consequently, a burden is placed on both the contractor and the Navy to work out an acceptable procedure, depth of penetration, and method of payment.

**SITE VISIT - October 1998 Drydock No. 6 Repairs and Inspection at PSNSY**

**Problem**

The Shipyard is concerned about the inconsistent quality of the concrete repairs and "excessive cracks" in the repairs recently completed on the headwall, phase 3 work order 0157, Drydock No. 6.

**Approach**

The Shipyard invited many individuals including the material supplier to visit the site, observe the work completed, observe a demonstration placement, and to participate in discussions. Douglas Burke of NFESC attended on October 22-23, 1998.

**Objective**

The Shipyard's objective is the have the contractor (Del-Jen) and the material manufacturer (Fosroc) discuss modifications to the methods and materials that will result in future repairs that have fewer cracks.

**Headwall Inspection**

The repairs at the headwall are complete. They are large and vary in size from 30 to 217 inches wide by 33 to 215 inches high by 6.5 to 10 inches thick. During a prior visit in August 1998, it was observed that the deteriorated concrete had been removed completely around the existing reinforcement, which is at about 12 inches o.c. with about 5 inches of cover to the surface. The cementitious repair material used was a prebagged mortar mix was Fosroc Rederoc LA. The concrete mortar was placed by the form and pump method about 30 days before this inspection. The forms were reportedly removed between 4 and 7 days after concrete placement, some of the cracks were reportedly visible at that time. The dock was then flooded and subsequently dewatered. On October 10, when the repaired areas were still wet, the cracks were photographed. The photos clearly show many cracks. No photos were made available for this report. The crack pattern is representative of differential drying shrinkage.

Cracks were measured on the headwall. Only the major cracks were visible on the day of inspection because the wall was dry and dusty. In contrast, the photographs show many other cracks that are narrower in width. The major cracks are about 12 to 20 inches apart in both directions. About a dozen cracks in various repairs were measured, their width varied from 0.002 to 0.005 inches. Acceptable industry standards permit 0.05 percent shrinkage at 30 days. The permissible crack width over a representative 16-inch crack-to-crack spacing is (0.0005) (16 inches)= 0.008 inches. This maximum allowable crack width is greater than any of the cracks measured. On average, the cracks in the headwall are about 0.02 percent at 30 days, an acceptable value.

**Laboratory Shrinkage Data**

Shrinkage tests conducted in the laboratory indicate that this specific repair product will shrink 0.013 percent in 30 days at 50 percent relative humidity. These values compare favorably to the measurements taken on site.

**Placement Demonstration**

During the demonstration the following important observations were made:

- The water monitoring tube on the mixer was new and marked with duct tape at a level that corresponded to 3.5 quarts of water per 55 pounds of dry mortar.
- A discarded water monitoring tube was discovered adjacent to the mixing machine. Duct tape had been used to mark a setting that was significantly higher than marked on the new tube. The old tube was caked with mortar and no longer transparent.
- The concrete substrate had not been flooded with water to saturate the substrate per the contractor's written procedures.
- The forms were not watertight.
- The mortar had no coarse aggregate in it.
- No vibration was used on the forms to consolidate the fresh concrete and apparently none is required because the consistence of the mixture is extremely fluid.
- The delivery hose used to supply fresh mortar from the pump to the form appeared to have a nominal 1.5-inch diameter and was about 200 to 300 feet long. This size may be too small to allow reliable delivery of the mortar from the pump to the form without clogging.
- A fresh concrete mixture of 3.5 quarts per 55 pounds of dry mortar was successfully pumped through the 1.5-inch hose to the form.
- When the form was filled up 5 feet, an experiment was conducted to slightly reduce the water content by an unknown amount. The purpose was to allow for an evaluation of the hardened concrete to determine if there were fewer cracks associated with the reduced water content. However, soon after the water was reduced the hose clogged and the experiment terminated.
- No slump measurements were made and apparently there are no quality control procedures established to document that consistent and high quality concrete is being mixed and delivered to the forms.

**Performance Exceptions**

Cracks permit the premature ingress of salt water that will ultimately result in the deterioration of the concrete and the steel reinforcement. All of the cracks will continue to increase in width as the repair material continues to hydrate over the next year. Consequently, every effort should be made to reduce crack frequency and width on future repairs.

## Differential Drying Shrinkage Cracks

- In general, the degree to which differential drying shrinkage and associated cracking can be minimized is improved by using a concrete mixture that contains the correct gradation of aggregates, and a size of coarse aggregate appropriate for the thickness of the repair and the placement method. The repair area would shrink less if it contained a coarse aggregate. The Renderoc LA does not contain coarse aggregate. Repairs greater than 4 inches thick should use a well graded 1-inch minus aggregate conforming to ASTM C33. Ready mix concrete is an acceptable substitute to the prebagged material used.
- The substrate surface should be completely saturated with clean water at least 24 hours prior to placement and then allowed to surface dry (saturated surface dry).
- Ensure that all of the forms remain in place for a minimum of 7 days. In addition, in hot conditions it is desirable to keep the forms wet during the entire 7-day curing period to minimize the water loss from the concrete. Apply two coats of a curing compound immediately after form removal.

## Conclusions

The cracks in the headwall repairs are due to differential drying shrinkage. They will ultimately have an adverse effect on the life expectancy of the repairs and the drydock. The crack frequency and widths are within the manufacturer's and contract specifications. However, it is feasible for the contractor in collaboration with the material manufacturer to produce future repairs that contain fewer cracks. The group discussions and placement demonstration should provide the contractor and the material representative with sufficient information to allow them to formulate a procedure for future repair work that will result in repairs that contain fewer and smaller cracks.

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8. Naval Facilities Engineering Service Center. Technical Report: Technology assessment of cathodic protection of reinforced concrete in the marine environment, by D. Burke and R. Jamond. Port Hueneme, CA (in publication).

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# CHAPTER 11

## UNDERWATER CONCRETE INSPECTION AND REPAIRS

**The following section contains excerpts from NAVFAC P-990 for underwater concrete inspection and repairs. This document is available through Stanley Black, NFESC, Phone: 805-982-1002; e-mail blacksa@nfesc.navy.mil.**

Naval Facilities Engineering Command
200 Stovall Street
Alexandria, VA 22332-2300

# NAVFAC P-990

for CD January 1997


# CONVENTIONAL UNDERWATER CONSTRUCTION AND REPAIR TECHNIQUES

Distribution limited to U.S. Government agencies only; Administrative/Operational Use; May 1995. Other requests for this document must be referred to the Naval Facilities Engineering Command.

• Diver assistance. ROVs can be used in conjunction with diver operations to provide useful functions including:

- Video observation of the diver/work area for safety and work quality documentation
- llumination of the work site
- Delivery of messenger lines and small tools
- Backup communication via video/slate

## 2.10 CONCRETING

Concrete placed underwater is the most widely used repair method in marine applications. The concrete mix should be designed in accordance with ACI 211.1-91, Standard Practice for Selecting Proportions for Normal, Heavyweight and Mass Concrete. Naval Facilities Engineering Command Guide Specification NFGS-03300G, Cast-in-Place Concrete, provides a number of concrete mix examples that are helpful in initial selection of trial mixes. The mix must have good workability and, thus, should meet the following conditions:

• The mixture must incorporate the proper proportions of sand and gravel in a rich paste of portland cement and freshwater.

• The total amount of mixing water should not exceed a water/cement ratio (w/c) of 0.44, i.e., 0.44 pounds of water for each pound of cement (this is the equivalent of 5.0 gallons of water per bag of cement). (A bag of cement weighs 94 pounds in the United States.) The "added" water is the net water to be included in the mixture after adjustments are made for the wetness or dryness of the aggregates. If the aggregates have free moisture (e.g., sand frequently has 5 to 6 percent by weight of free water), this becomes part of the mixing water so that less added water is needed. On the other hand, if the aggregate particles are surface dry and not saturated, they will absorb some of the gross mixing water (e.g., up to 1 percent or so of aggregate weight as is often the case with coarse aggregates); therefore, extra mixing water must be added.

• The mixture should contain not less than 7 bags and not more than 10 bags of cement per cubic yard of concrete. The cement should be ASTM Type II with a tricalcium aluminate content greater than 5 percent and less than 10 percent. Pozzolans (fly ash) conforming to ASTM C618 are a good addition to the mix. Fly ash can have the beneficial effects of improving the durability of the concrete, increasing corrosion protection to the reinforcing steel, and improving workability of the mix. The amount of fly ash used typically replaces about 15 to 20 percent of the weight of cement in the mix. The total weight of the cement and fly ash is considered the cementitious content of the mix.

• The concrete should incorporate a water-reducing admixture or a high-range water reducing admixture (superplasticizer). Superplasticizers allow concrete to have high workability and, at the same time, low water content. Do not exceed the recommended manufacturer's dosage as segregation may occur.

Concrete to be used in the intertidal zone and the marine atmosphere that will be subjected to subfreezing temperatures should be air-entrained with an air-entraining admixture in accordance with:

- CEL CR 81.009, Survey of Techniques for Underwater Maintenance/Repair of Waterfront Structures (and Amendment P00001)

- NCEL TN-1624, Underwater Inspection of Waterfront Facilities: Inspection Requirements Analysis and Nondestructive Testing Technique Assessment

- PCA, Design and Control of Concrete Mixtures, of the American Concrete Institute

- The admixture manufacturer's instructions

A relatively new type of admixture is being marketed called an anti-washout admixture. Anti-washout admixtures are intended for use in concrete placed underwater. The admixture increases the viscosity of the concrete mix, which improves the resistance to water dilution and washing away of cement fines in the surface exposed to water. High-range water reducers and anti-washout admixtures are usually used together. However, not all commercially available admixtures are compatible with all brands of cement and air-entraining admixtures. Therefore, if possible, all admixtures should be from the same manufacturer and their compatibility with the cement being used should be confirmed. The quantities used should be in accordance with the manufacturer's instructions. Admixtures are usually dissolved in the mixing water before it enters the concrete mixer. Admixtures should be used in accordance with ACI 212.3R-91, Chemical Admixtures for Concrete, and ACI 212.4R-93, Guide for Use of High-Range Water-Reducing Admixtures (Superplasticizers) in Concrete.

- The sand and gravel should be physically sound. Great care should be exercised in selecting aggregates at locations where sound material is not readily available, particularly coral and limestone islands. Concrete made with coral or limestone may not be durable. If coral or limestone must be used, thoroughly washed it with freshwater to remove all organic and clay-like fine material. For reinforced concrete, wash the aggregate with freshwater to remove all salts.

- The formwork in which the concrete is placed must be secure, carefully fitted (so that there are no leaks at joints and corners), and designed so that no underwater currents can pass through it. Provision must be made for the seawater displaced by the concrete to escape from within the form.

- Low temperatures during mixing and curing of concrete (i.e., below 50°F) can delay strength development. Chemical admixtures, without chlorides or hot water, can be used in the concrete to accelerate early strength gain. In reinforced concrete, calcium chloride should not be used as an accelerator; however, for unreinforced concrete, calcium chloride is acceptable.

- An enclosed chute or "trunk" should be used and the end of the chute should be kept embedded in the newly deposited concrete so that there is no washing away of cement fines or mixing with water during placement.

Concrete placed underwater is typically placed in a form. Forms used in UCT operations include cut plywood forms that are fabricated on site, flexible fabric forms that are secured underwater, cylindrical fiberglass forms, rigid fiberboard forms, cut metal pipe, as well as others.

When calculating the quantity of concrete needed, always be liberal to ensure that a sufficient amount is available to complete the job. Unplanned interruptions of an underwater concrete placement operation can have disastrous effects on the quality of the completed work. A complete description of working with concrete forms is given in Chapter 5, Underwater Maintenance and Repair Procedures.

Concrete is placed underwater by four methods: pumping, tremie, prepacked, and sacked. By far the most common method used by the UCTs is pumping. When a concrete pump is not available, the tremie method is used as a substitute. For special circumstances where the pumped or tremie method is not suitable, the prepacked method may be used. Sacked concrete may be used to build underwater walls, protect slopes from underwater wave and current wash, and cover cables and pipelines. Each of these methods is described below.

### 2.10.1 Pumped Concrete

Pumping freshly mixed concrete is the most expeditious means of placing concrete underwater in spaces of limited accessibility. It is generally preferred over the tremie method, however, the following considerations should be accounted for:

- Pumped concrete should be used to fill the forms from the bottom upward, displacing the seawater as additional concrete is forced in at the bottom. (Recently, new admixtures have been developed that enable concrete to be dropped without a tremie.)

- High quality concrete is required, because the mixture must be both workable and cohesive, so as to pass through the pump without blockage.

- Workable mixtures containing relatively small coarse aggregate particles tend to provide an easily placed concrete.

- The slump must be carefully controlled to prevent segregation, as excessively wet mixtures will sometimes segregate and cause blockage in the hose or pipeline. (High-range water reducers used as an admixture in the concrete can provide the high workability and eliminate segregation.)

- Coarse aggregate should consist of rounded particles, as crushed stone mixtures are comparatively difficult to pump because the angular particles tend to increase friction in the pipeline.

- If it is necessary to use crushed rock, the maximum coarse aggregate size should not be greater than one-third the smallest inside diameter of the hose or pipe.

- The properties of the fine aggregate (sand) are more important than those of the coarse aggregate. In particular, the sand should have a relatively high proportion of the finer sizes (i.e., 15 to 30 percent should pass the no. 50 sieve, and 5 to 10 percent should pass the no. 100 sieve).

- Porous aggregates (e.g., expanded clay, foamed slag, pumice, and many coralline materials) should be avoided unless denser aggregates are not available. If porous aggregates are used for pumped concrete, they should be presoaked as described in ACI 304.2R-91, Placing Concrete by Pumping Methods.

- Inserting a pig between mixes is usually a good idea. Care must be taken with long vertical drops with the pump hose. With no backpressure, i.e., break seal or disconnect, the concrete wants to siphon and collapse hose, creating an instant plug.

The pumping method offers several advantages:

- Concrete can be transferred from a barge directly into the forms.

- The process is less subject to operator error than the tremie concrete process.

- Pumped concrete is more easily placed in awkward situations and locations with difficult access, such as under piers.

- The pressurization process consolidates the repair material, providing for full encapsulation of exposed reinforcing steel.

The concrete pump most often used by the UCTs is a direct-flow, reciprocating piston pump with a pumping rate of about 25 $yd^3/hr$, a hopper capacity of 6 $ft^3$, and a maximum pumping distance of about 400 feet. The pump, illustrated in Figure 2-54, is normally fitted with a 2-inch-diameter hose but can be fitted with a 2-1/2-inch-diameter hose for mixes that are hard to pump. It requires concrete with 3/8-inch maximum size aggregate, graded as indicated in Table 2-24. A recommended concrete mix design for pumped concrete that has proved successful in the past is given in Table 2-25. While this is suitable for placement in areas of limited accessibility, larger aggregate sizes and corresponding pump sizes should be used for large pours.

Carefully planning the location of the pump and hose routing before starting an operation can save subsequent moves throughout the project. The pipeline should be either horizontal or vertical rather than inclined, wherever possible. With an inclined pipeline, any water bleeding from the freshly mixed concrete within the pipeline will collect above the concrete and run down the inside of the pipe.

When placing pumped concrete, surfaces that might trap air are trimmed, or vent pipes are provided in the formwork.

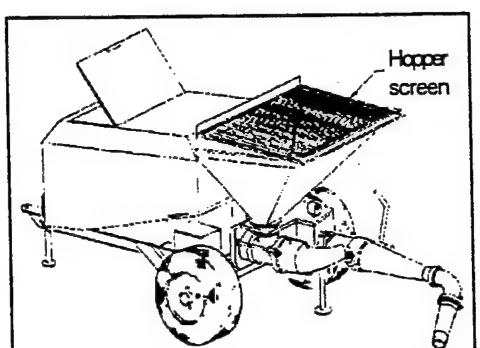



Figure 2-54. Typical concrete pump.

Table 2-24
Gradation of Aggregates for Pumped Concrete

| Aggregate | U.S. Standard Sieve | Percent Passing (by weight) |
|---|---|---|
| Gravel | 1/2 in. | 100 |
| | 3/8 in. | 85 to 100 |
| | No. 4 | 10 to 30 |
| | No. 8 | 0 to 10 |
| | No. 16 | 0 to 5 |
| Sand | 3/8 in. | 100 |
| | No. 4 | 95 to 100 |
| | No. 8 | 80 to 100 |
| | No. 16 | 50 to 85 |
| | No. 30 | 25 to 60 |
| | No. 50 | 15 to 30 |
| | No. 100 | 5 to 10 |

Table 2-25
Design Mix for Pumped Concrete for Underwater Use
[Quantities indicated are per cubic yard of concrete.]

| Item | Quantity |
|---|---|
| Cement (Type II) | 640 pounds (8 sacks) |
| Fly Ash | 112 pounds |
| Sand | 2,100 pounds |
| Gravel (3/8 inch maximum) | 800 pounds |
| Water | 40 gallons |
| Admixture - Water Reducer | In accordance with manufactuer's recommendations |
| Slump | 6 inches |
| Strength | 4,000 psi |

(A rough profile on the concrete substrate resulting from surface preparation of existing concrete prior to a repair will not entrap air.) The formwork is designed to accommodate the weight and pressure of the concrete. The minimum design pressure is 14 psi. The maximum pressure occurs when the formwork cavity is full and pressurized. The pump hose is attached to the formwork with flanged plumbing fittings and ball valves, or the pump line is attached using hand-held friction fit insertion followed by wooden plugs.

The pumping sequence is from bottom to top. Large repairs may require bulkheads to separate placements into manageable areas. Pumping continues until the concrete flows out of the top of the form or through a vent. Once the form is full, pump pressure is exerted on the concrete, causing it to consolidate and make intimate contact with the form, or existing substrate in the case of a repair.

Before concrete is discharged into the hopper, 3 to 4 gallons of water should be sprayed into the hopper followed by about 5 gallons of a creamy cement and water slurry (1/2 bag of cement to 5 gallons of water). This procedure lubricates the hose and prevents separation and blockages. Delays as long as 1-1/2 hours can generally be tolerated if the mixture is moved several feet at least every 5 minutes (while in the hose or pipeline) until continuous pumping is resumed. If the concrete becomes excessively stiff because of a long delay, the concrete should not be retempered by adding water but should be discarded and a fresh batch mixed. If the concrete contains a high range water reducer (superplasticizer), the slump can decrease substantially after 3/4 to 1 hour. Concrete in the mixer can have additional plasticizers added to increase the slump one time. Concrete in the pipeline should be removed before the slump decreases. When concreting is completed, all parts of the pump and pipeline should be thoroughly cleaned.

The concrete should be pumped as near to its final underwater position as possible. The diver who has control of the discharge end should not permit lateral flow within the open-top form of more than 2 or 3 feet. The discharge end of the line must always be kept buried in the mass of fresh concrete; otherwise, washout will occur at the point where the concrete comes out. Aluminum pipe should not be used because an adverse chemical reaction with the concrete may occur. The pipeline should be protected from any excessive heat (solar included).

**CAUTION**

Use extreme caution when a hose blockage requires opening a coupling due to the pressure from the concrete column.

### 2.10.2 Tremie Concrete

One method of placing concrete underwater, especially at easily accessible locations, involves a tremie -- a steel tube with a hopper for filling at its upper end. A plug, consisting of either a rubber ball or a wad of burlap that fits snugly inside the tremie, is inserted below the loading hopper. The freshly mixed concrete, introduced at the hopper, forces the plug down and displaces the seawater. The tremie is continually replenished with concrete while the lower end is kept embedded in the newly deposited concrete. Compaction by vibration is not permitted, as it generally results in excessive

laitance. Tremie concrete must be quite workable so that it flows readily into place. Fine aggregate contents of 45 to 55 percent by volume of total aggregate, and air contents of 4 to 5 percent, are generally desirable. A typical aggregate gradation is indicated in Table 2-26. A typical mix design for tremie concrete is given in Table 2-27. The size of the coarse aggregate should be restricted as follows:

- Very large pours: 1.5 inch maximum
- Normal pours: 0.75 inch maximum
- Restricted access pours: 0.375 inch maximum

It is general practice to use a steel tremie, but a rigid rubber hose could be substituted. Joints in the tremie must be well-gasketed and sealed, otherwise the water could leach the cement from the concrete. An aluminum alloy tremie should not be used because an adverse chemical reaction could occur, producing inferior concrete. For deep placements, the tremie should be fabricated in sections, with joints that allow the upper sections to be removed as the placement progresses.

The size of the tremie depends on the maximum size of gravel and on the quantity of concrete to be placed; the usual range in diameter is from 4 to 10 inches, and should be at least eight times the maximum coarse aggregate size. The slump of tremie concrete should be maintained between 6 and 9 inches.

The quality of tremie concrete is greatly dependent on proper mix design and placement. Reinforcing should be designed to minimize segregation due to a screening effect. The largest size bars possible should be used, and they should be spaced a minimum of three to four times the aggregate size. Admixtures are used to improve the flowability of the concrete. Otherwise, much laitance, due to washing out of cement, and gravel pockets, due to lifting of the tremie pipe to facilitate placement, can result. Water reducing retarders in combination with an air entraining agent, and/or superplasticizers, are used to provide increased flowability at low water cement ratios. This increased flowability permits wider spacing of the tremie pipes, longer pipe lengths, wider pipes, and longer placing times.

The tremie method of pumping concrete is best suited for footings. The tremie is difficult to handle and will often hang up in spacers typically used in the encasement of piles.

### 2.10.3 Prepacked Concrete

Prepacked concrete is used on large underwater repair jobs where placement of regular concrete would be either difficult or impossible, and/or where minimization of shrinkage is a requirement. It is most economical for isolated locations, where pumping distances are excessive.

Prepacked concrete involves placing coarse aggregate in a form and then filling the voids in the aggregate mass with grout, as shown in Figure 2-55.

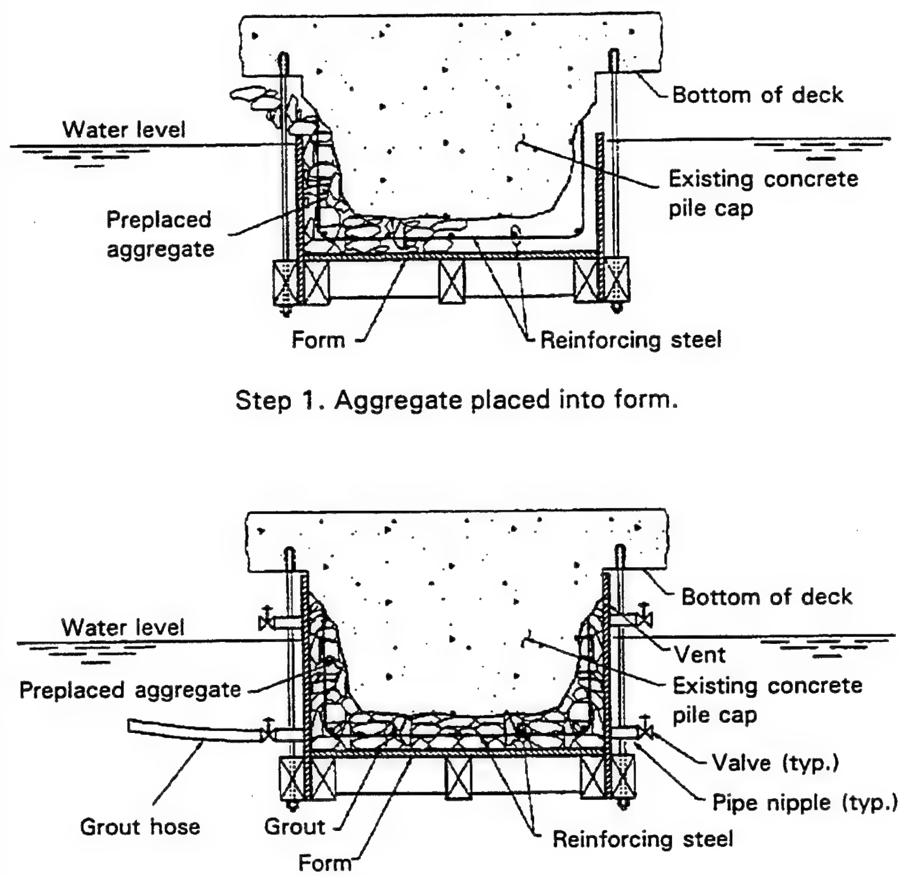
The preplaced aggregate concrete components should meet the requirements specified in Chapter 7 of ACI 304.2R-91. Form materials are essentially the same as for regular cast-in-place concrete, except that they must be sufficiently tight to prevent leakage.

Table 2-26
Gradation of Aggregates for Tremie Concrete

| Aggregate | U.S. Standard Sieve | Percent Passing (by weight) |
|---|---|---|
| Gravel | 1/2 in. | 100 |
| | 3/4 in. | 90 to 100 |
| | 3/8 in. | 20 to 55 |
| | No. 4 | 0 to 10 |
| | No. 8 | 0 to 5 |
| Sand | 3/8 in. | 100 |
| | No. 4 | 95 to 100 |
| | No. 8 | 80 to 100 |
| | No. 16 | 50 to 85 |
| | No. 30 | 25 to 60 |
| | No. 50 | 10 to 30 |
| | No. 100 | 2 to 10 |

Table 2-27
Design Mix for Tremie Concrete for Underwater Use
[Quantities indicated are per cubic yard of concrete.]

| Item | Quantity |
|---|---|
| Cement (Type II) | 640 pounds (8 sacks) |
| Fly Ash | 112 pounds |
| Sand | 1,300 pounds |
| Gravel (3/8-inch maximum) | 1,800 pounds |
| Water | 40 gallons |
| Admixture - Water Reducer | In accordance with manufacturer's recommendations |
| Slump | 6 inches |
| Strength | 4,000 psi |




Step 1. Aggregate placed into form.

Step 2. Form completed and grout being placed.

Figure 2-55
Steps for placing prepacked concrete into form.

The cement, aggregates, and water for preplaced aggregate concrete are also similar to those for regular cast-in-place concrete, however, the concrete mix contains a higher percentage of coarse aggregate. Aggregate gradation is in accordance with Table 7.2 of ACI 304.2R-91. A typical gradation is indicated in Table 2-28. For repair depths exceeding 12 inches, gradations with larger size aggregates are used. The diameter ratios of the smallest particles of coarse aggregates to the largest particles of sand should not be less than 4 to 1.

The void content of the coarse aggregate is as low as possible, which can usually be attained by grading it uniformly from the smallest aggregate size to the largest aggregate size. The gradation shown in Table 2-28 is recommended to minimize the void content. Gap grading sometimes provides a lower void content than uniform grading. The void ratio of the cavity after the aggregate is placed is usually 35 to 40 percent.

Table 2-28
Gradation of Aggregates for Prepacked Concrete

| Aggregate | U.S. Standard Sieve | Percent Passing (by weight) |
|---|---|---|
| Gravel[a] | 2 in. | 100 |
| | 1-1/2 in. | 90 to 100 |
| | 1 in. | 20 to 55 |
| | 3/4 in. | 0 to 15 |
| | 3/8 in. | 0 |
| Sand[b] | No. 8 | 100 |
| | No. 16 | 95 to 100 |
| | No. 30 | 55 to 80 |
| | No. 50 | 30 to 55 |
| | No. 100 | 10 to 30 |
| | No. 200 | 0 to 10 |

[a]Placed separately before intrusion of grout.
[b]Used in the grout

The grout is proportioned in accordance with the requirements of ASTM C938, Standard Practice for Proportioning Grout Mixtures for Preplaced-Aggregate Concrete. Its consistency is determined in accordance with ASTM C939, Flow of Grout for Preplaced-Aggregate Concrete (Flow Cone Method). The consistency, which is measured as an efflux time, should be on the order of 22 $\pm 2$ seconds. The grout consists of Portland cement, sand, and Class F fly ash of Class N pozzolan. Whether fly ash or pozzolan is used will largely depend on the Contractor's previous experience and/or the results of trial tests. Generally, the mix that provides the best workability, highest strength, and least shrinkage should be recommended for repairs.

Class F fly ash is normally produced from burning anthracite or bituminous coal, while Class N pozzolan is a raw or calcined natural material containing silica or aluminum in a finely divided reactive form. The fly ash and pozzolan are used to improve pumpability of the grout, extend its handling time, and provide increased impermeability and erosion resistance.

The fly ash and pozzolan must conform to the requirements of ASTM C618, Specification for Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Portland Cement Concrete. The proportion of portland cement to pozzolan is in the range of 2.5:1 to 3.5:1, by weight, as discussed in ACI 304.2R-91. The ratio of cementitious material (i.e., Portland cement plus pozzolan or fly ash) to sand is approximately 1:1, while the water to cementitious material ratio ranges from 0.42 to 0.50.

In addition, a grout fluidifier, meeting the requirements of ASTM C937, Standard Specification for Grout Fluidifier for Preplaced-Aggregate Concrete, is used to offset the effects of bleeding, reduce the water/cement ratio, and retard stiffening. The dosage of

fluidifier is approximately 1 percent of cementitious material by weight. The amount of bleeding in the grout should be less than the amount of expansion, as determined in accordance with ASTM C940, Standard Test Method for Expansion and Bleeding of Freshly Mixed Grouts for Preplaced-Aggregate Concrete in the Laboratory.

The forms are constructed in two steps, as shown in Figure 2-55. At the completion of the first step, there must be sufficient openings to place the coarse aggregate. After washing and screening the coarse aggregate so that it is free of fines and bond inhibiting materials, it is placed in the forms using a flexible elephant trunk or rubber-lined chute to avoid segregation and breakage. The forms must be strong enough to permit the aggregate to be tamped with vibrators or tampers in order to minimize the void content. When coarse aggregate is to be placed in restricted areas of forms, the aggregate particles are frequently moved by hand. Movement can alternatively be accomplished by the use of an air lance, utilizing a short length of 3/8-inch pipe.

Once the aggregate is in place, the formwork is completed, as shown in Step 2 of Figure 2-55. The forms are built sufficiently tight to prevent grout leakage. The aggregate is kept wet until the grout is placed to facilitate the grout's smooth flow. This requires caulking of all small openings and sealing adjacent form panels with tape.

The grout pipes are installed, generally before aggregate placement, and often are fixed to the form or to a reinforcing cage. The pipes range from 3/4 inch to 1 inch in diameter, and are spaced at 5 to 6 feet on center. After the aggregate is placed, a highly flowable structural grout is pumped into the bottom of the form and flows upward, displacing the water.

The grout makes contact with the existing concrete surface as the cavity is filled, providing intimate contact and bonding. A unique advantage of this method is the point-to-point contact between the coarse aggregate, resulting in a drying shrinkage of approximately one-half that of regular cast-in-place concrete. The aggregate contact restricts the volume change of the cement grout as drying shrinkage occurs. Densities of preplaced aggregate concrete are generally slightly higher than those of normal concrete. Bond strengths of prepacked to regular concrete are between 70 to 100 percent of that attainable in regular concrete. This makes it possible to restore deteriorated concrete members to near their original strengths or to enlarge existing members to take additional loads.

When repairing existing components, weakened material should be removed to expose sound concrete, and the surfaces of sound concrete should be roughened by either chipping or water blasting before repair. Space must be provided for at least 3 to 4 inches of new prepacked concrete. Grouting should begin immediately after the forms are completed to avoid contamination of the aggregate. Grout placement always starts from the lowest point in the forms. After grout flows from adjacent ports, the grout hose is disconnected from the port being pumped and reconnected to the port showing new flow, while the previous port is sealed. The process continues until the cavity is full and pressurized. When the forms are filled, a closing pressure of approximately 10 psi is held for up to 1 hour to drive out all air and water through a vent at the highest point. The forms may be removed 1 or 2 days later.

When handling prepacked concrete, it is important to:

- Prevent fines from collecting in the coarse aggregate because they tend to impede the flow of the grout. These fines, which could result from abrasion of the coarse aggregate during handling, collect on the bottom of the conveying barges or trucks.

- Deposit the aggregate in a clean place that is free of mud, silt, slurry, or other contamination.

- Pump the grout promptly after aggregate placement.

- Protect the aggregate from contamination between the times of placement and grout intrusion.

### 2.10.4 Sacked Concrete

Slope protection, scour protection around structures, and cover for underwater cables and pipelines can sometimes be done by using sacked or bagged concrete. Three methods are used:

- In one method burlap sacks are half-filled with dry mix. The sacks are lowered underwater on pallets, then placed by a diver. Water penetrates the burlap and concrete to imitate hydration of the cement. This method has one advantage in that the time of handling and placing is not critical, but the sacks can be dislocated by waves or other water movement before the concrete sets. The bond between adjacent sacks may or may not be thorough, and the cement may not be uniformly distributed through the mix. The concrete mix used should be similar to that used for tremie concrete, but with the water and admixtures omitted.

- Another method is to use a wet mix in burlap or jute sacks. The mix should be low-slump. Bags of 1-$ft^3$ capacity are filled to about two-thirds full, securely tied, then promptly lowered into position and placed. Good bond can be obtained and the general quality of the mix can be verified. Bags can be placed in stretcher-and-header courses so as to interlock. The concrete mix used should be similar to that used for tremie concrete, but with less water.

- The preferred method is to fill the sacks underwater. Fabric forms of nylon can be placed over the surface to be protected and pumped full of concrete. The fabric is porous and acts as a combined filter and protective lay. The concrete mix used should be similar to that used for pumped concrete.

## 2.11 GROUTING

### 2.11.1 Materials

Grouts for underwater use can be generally classified as either a hydraulic cement or an epoxy. Several variations of both the hydraulic cement and the epoxy are commercially available for use in different applications.

A hydraulic cement is a single-component cement that is capable of setting and hardening underwater because of the interaction of water and the constituents of the cement. Admixtures are available from hydraulic cement manufacturers for obtaining specific performance goals (i.e., accelerate or slow down the reaction rate).

Epoxy grouts are used for bonding different surfaces or for filling thin voids. All epoxy grouts have at least two components: a resin component and a hardener component. Some epoxy grouts also have additions of fillers or aggregates to "extend" or increase the usable volume of the grout or to modify the characteristics of the grout. Epoxy grouts without additions or fillers are called "neat" grouts. Epoxy grouts are commercially available in different formulations, each having a specific performance or physical characteristic (i.e., strength, mixing ration, pot life, moisture sensitivity, etc.).

Table 2-29 lists the materials for the grouting applications described in Sections 2.11.2 through 2.11.5.

Table 2-29
Grouting Applications and Materials

| Procedure | Material |
|---|---|
| Securing U-bolts or rock anchors to seabed | • Hydraulic cement grout<br>• Neat epoxy resin grout |
| Repairing deteriorated concrete surfaces | • Epoxy resin/oven-dried aggregate grout<br>• Hydraulic cement grout |
| Repairing cracks in concrete | • Neat epoxy resin grout |
| Installing anchor bolts in concrete | • Prepackaged epoxy grout<br>• Hydraulic cement grout<br>• Neat epoxy resin grout |

2.11.2 Securing U-Bolts or Rock Anchors to Seabed

Grouted-in-place U-bolts or other types of anchor bolts provide an excellent anchoring technique for pipelines or for split-pipe cable protection systems in soft rock or coral seabeds. Either hydraulic cements or neat epoxies are used for grouting these anchor bolts.

The first step in securing the U-bolt or rock anchor is to drill a hole of the required dimensions (usually 1 to 1-1/2 inches in diameter, 12 to 16 inches deep) at the desired anchoring location. See Section 2.2.3 for a discussion of drilling equipment and techniques. In some locations, drifting sand can be expected to fill the drilled hole before the grout can be inserted. In this case, compressed air should be used to blow the hole free of sand just prior to inserting the grout. If compressed air is not available, the hole can be temporarily plugged by placing a pipe into the hole immediately after drilling. The diameter of the pipe should be slightly smaller than the diameter of the hole and the upper end of the pipe should extend 6 inches to 1 foot above the seabed to keep the sand out.

### 2.11.2.1 Hydraulic Cement

Hydraulic cement is placed by either of two methods. In the surface mix method, the cement-sand mixture and water are poured into a 4- to 5-foot length of flexible plastic tube, about 5 inches in diameter and 6 mils thick, with a knot in the lower end. The mixture is shaken and kneaded on the surface, the tube is carried to the bottom by a diver, and the contents are squeezed into the drilled hole. This procedure has often proved to be unsatisfactory because the mixture tends to harden too rapidly. A better solution, referred to as the toothpaste-tube method, is to pour the cement-sand mixture into a tied-off tube, twist and hold the tube at the center, pour in the freshwater, and then tie off the top of the tube. The tube is then lowered to the diver in a tool bag, who releases the twist in the tube and mixes the grout on the bottom. This will allow sufficient time to squeeze the grout into the hole before the mixture hardens.

When working with hydraulic cement it is prudent for the diver to have voice communications (COMS) with the surface. Using COMS, the diver can inform the tenders when to lower tubes of cement to the worksite.

### 2.11.2.2 Epoxy Resin

When selecting an epoxy resin material, the water temperature at the work site as well as the manufacturer's packaging and application technique should be considered. Generally, for applications where the water temperature is greater than 60 degrees, a polyester resin approved for marine use may be used. However, for temperatures between 23 and 60°F, a vinylester resin (or other specially formulated resin) should be used. The vinylester resin can also be used for warmer areas.

Generally, epoxies consist of two components (resin and catalyst) that may come in a variety of different mixing ratios. One manufacturer's product (HILTI) comes packaged in a self-contained glass vial that is inserted into a hole (made by a rock drill) and then mixed by rotating a threaded fastener down into the glass vial. The hammering/rotary action applied by the drill breaks the glass vial and mixes the components of the capsule together. The advantage of this particular system is that the mixing technique minimizes the amount of handling. However, because the mixing takes place in the fastener hole, it is important that the hole fully contain the epoxy during the mixing process. For this reason, the use of glass vial products is not recommended in coral seafloors (where the epoxy may not be well contained during the mixing process).

For applications requiring flexibility in the amount of grout dispensed, the epoxy is placed by using either a caulking gun or the NFESC grout dispenser. Both of these techniques have been used in coral seafloors, where it has been shown that increased fastener holding strength can be achieved by completely filling all of the void spaces in a fastener hole with epoxy (effectively strengthening the foundation material).

Pneumatically-operated caulking guns (60 to 100 psi) that dispense epoxy grouts from disposable plastic cartridges are commercially available.

The grout is usually premixed in buckets on the surface, poured into the cartridges, and then relayed to the divers at the work site. The divers then systematically load and dispense the contents of each cartridge. Since most epoxy grouts have a relatively short pot life (30 minutes or less), successful application using this technique requires quick execution and teamwork.

The NFESC grout dispenser, shown in Figure 2-56, is a pneumatically-powered diver tool. The grout dispenser simultaneously mixes and dispenses neat epoxy (1:1 mixing ratio) from two 20-fluid ounce disposable plastic cartridges. The tool can be reloaded underwater with full cartridges of epoxy and can dispense epoxies at a repetitive rate of 1,000 fl oz/min. In addition, the tool incorporates a "blow-out" system for cleaning residual epoxy from the mixing mechanism. This helps prevent epoxy from hardening inside the tool. The tool weighs approximately 35 pounds dry and 8 pounds submerged. In comparison with other underwater grouting techniques, this method delivers larger quantities of grout in shorter periods of time, employs air pressure rather than muscle to mix and dispense epoxy, and eliminates errors in mixing.

Also, if desired, the epoxy components can be preassembled (by the epoxy manufacturer, cartridge manufacturer, or a contractor), thereby minimizing exposure or contact with dive personnel and equipment.

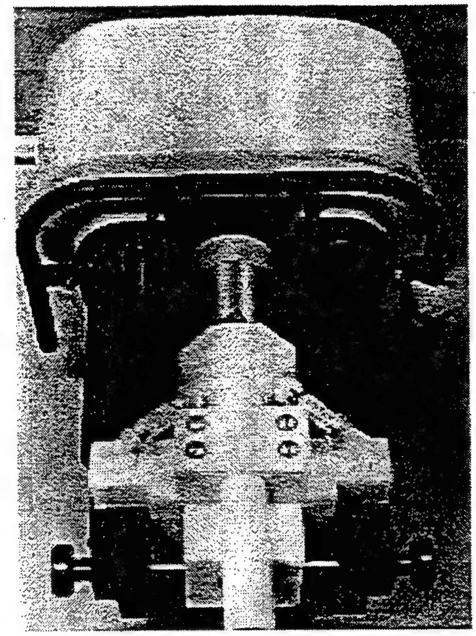


Figure 2-56
Diver-operated epoxy grout dispenser.

### 2.11.3 Repairing Deteriorated Concrete Surfaces

For repairing deteriorated underwater concrete surfaces, either epoxy resin/oven-dried aggregate mortar, Portland cement mortar, or specially formulated prepackaged concrete mixes are used. If epoxy mortar is used, the epoxy resin must be formulated for beading to wet surfaces and underwater application. The concrete surfaces to which the mortar is to be applied must be cleaned by sandblasting or waterblasting to allow for good bonding action. Loose concrete must be chipped out and corroded reinforcing bars cleaned and supplemented by new bars if necessary. The mortar should be mixed and applied in accordance with the manufacturer's directions. The mortar can be applied either by hand or tool smearing.

The prepackaged concrete mixes can be either hand applied or pumped. Surface preparation is as described above for the mortar. The special formulation provides dimensional stability underwater with minimal washout.

### 2.11.4 Repairing Cracks in Concrete

The best method for repairing small to medium cracks in concrete piles or structures is pressure injection of neat epoxy resin. This method can generally be used for

cracks up to 1/4-inch wide. The epoxy selected should be a low-viscosity formulation suitable for wet surfaces and underwater application. In choosing the appropriate resin, it is important to confer with the manufacturer to ensure that the resin is compatible with the crack size and depth, temperature variations, characteristics of the concrete, and the equipment to be used to apply the resin. Holes should be drilled into the bottom of the crack every 6 inches to 3 feet along its length, and small tubes, or one-way polyethylene valves, should be installed. The area around the tube or valve and the entire surface of the crack should be sealed with a quick-setting epoxy paste adhesive. Fiberglass tape can be used in conjunction with the paste as described later in Section 5.7.2. The low-viscosity epoxy resin is then injected into the lowest valve until it reaches the level of the next valve. The lower valve is closed. The epoxy is then injected in the upper valve and the procedure repeated until the crack is filled. In the event of unanticipated leakage from previously undetected cracks, hydraulic cement is used as a sealer.

### 2.11.5 Installing Anchor Bolts in Concrete Structures

Using prepackaged epoxy grout in glass tubes is an extremely efficient method of installing anchor bolts in existing concrete structures. The action of screwing the anchor into the tube breaks the glass, which then acts as a coarse aggregate, and mixes the resin to start the setting action. This type of grouting system is very cost effective for small projects.

Alternatively, Portland cement mortar or epoxy resin can be used. With these materials the grout is squeezed or injected into the drilled and cleaned hole so that the hole is almost full. The anchor bolt is then inserted into the hole, forcing some of the grout out and ensuring that the hole is completely filled.

## 2.12 PROTECTIVE COATING APPLICATION

The life of marine structures can be extended when proper measures are taken to protect surfaces in contact with seawater. Applying protective coatings is the conventionally accepted method of protecting steel marine structures from corrosion attack. Since marine structures are subject to salt, air, or water exposure, the coating systems used are much more sophisticated than ordinary systems for inland steel structures.

Many references are available for applying marine structure protective coatings above the water line. Two excellent references are:

- NAVFAC MO-104 - Maintenance of Waterfront Facilities (scheduled for revision in 1997)
- NAVFAC MO-110 - Paints and Protective Coatings

The following discussion is concerned only with the application of protective coatings below the high water line.

### 2.12.1 Types of Protective Coatings

Several underwater-curing, epoxy-polyamide mastic coatings are available on the market. The suitability of these coatings for a particular application may be affected by factors such as seawater temperature and salinity, surface preparation, and electrical charge on steel surfaces (as from a cathodic protection system). Therefore, it is desirable to test the proposed coating on a small section of the surface to be coated to confirm that it will adhere properly.

The available underwater coatings fall into two general categories: brush/roller applied and hand applied. The hand-applied coatings are generally more suitable for UCT operations. They consist of two separate packages, each with a different color pigmentation. The viscous liquid epoxy resin in one package reacts with the viscous polyamide resin in the second package to form a hardened mass.

Apply a three-coat system with a total dry film thickness of 9 mils (0.009 inch) consisting of one coat of MIL-P-24441 formula 150 (0.003-inch minimum thickness), one coat of MIL-P-24441 formula 151 (0.003-inch minimum thickness), and one coat of MIL-P-24441 formula 152 (0.003-inch minimum thickness).

Coal tar epoxy polyamide is also used but has toxicity problems, primarily during application. The recommended coating of this type is Steel Structures Painting Council Paint #16.

All coatings should be mixed and applied following the manufacturer's instructions. It cannot be stressed too strongly that the manufacturer's instructions must be followed to obtain a long system service life. Shortcuts can reduce service life.

### 2.12.2 Surface Preparation

To prepare the surfaces to receive the underwater coating, all dirt, oil, grease, loose paint, rust, rotted and spalling concrete, rotted wood, marine growth, and other interference materials must be removed. Adhesion of the coating will only be as good as the cleanliness and soundness of the substrate. Sandblasting to a near white metal is the preferred cleaning method but high-pressure waterblasting may be acceptable. Scraping and other manual means of surface preparation are time consuming and of limited benefit.

In many cases, local environmental restrictions require that the abrasive blast media and paint be collected and not fall into the water. Abrasive blasting and the application of some paints may also be locally regulated. Particular care should be taken when removing lead-based paint. When removing lead-based paint, the worksite generally needs to be enclosed and all of the paint that is removed needs to be treated as hazardous waste. In addition, special precautions must be taken to prevent contact with or inhalation of the old paint or contaminated blast media.

Metal surfaces should be coated as soon as possible after cleaning to minimize the formation of new corrosion products. If the surface is not coated within about 4 hours, it should be cleaned again.

Field experience has shown that successful application of underwater coatings on steel surfaces requires that the steel have a positive or neutral charge. A negative charge

on a steel surface will repel the negatively charged epoxy coating. An underwater voltmeter should be used to check the charge on the surface. If the structure has a cathodic protection system, it can be neutralized either by turning off the system, as in the case of an impressed current system, or by removing remnants of a sacrificial anode system..

### 2.12.3 Mixing

The two differently colored components of the underwater coating should be thoroughly mixed (preferably in 1-quart quantities) by hand to produce a uniform color. The preparation and method of mixing depend on the particular product being used; therefore, the manufacturer's instructions should be carefully followed.

> **CAUTION**
> Since some people may be sensitive to epoxy or polyamide resins, wear protective gloves.

Most underwater coatings have a work time, or pot life, ranging from 15 to 45 minutes at 70°F. The pot life is usually reduced by 50 percent at temperatures above 80°F. Some preplanning, therefore, must be given to the quantity of material that can be used in the span of the pot life. Material may still be workable after the pot life has passed, but it will not adhere properly.

### 2.12.4 Application

Epoxy coatings are best applied by hand. A ball of the thoroughly mixed coating should be picked up with wet gloved hands and pressed against the area to be coated. It should be forced from the center to the outside to give a thickness of 1/8 to 1/4 inch and then smoothed and feathered at the edges. Epoxies are best applied at temperatures above 60°F. An average experienced diver can apply the coating at a rate of about 1/2 $ft^2$/min. Based on a pot life of 15 minutes, not more than 1/2 gallon should be prepared at one time for each diver.

Regarding paints for potable water tanks, yes, potable water tanks can be painted. Specific paints are approved by each State for use in potable water tanks. Most approved paints are epoxies. The National Sanitation Foundation has several approved systems.

### 2.12.5 Coverage

The coverage of underwater coatings varies with each specific material and the applied thickness. For a 3/16-inch thickness, theoretical coverage is 8.5 $ft^2$/gallon, practical coverage is about 6 $ft^2$/gallon.

slate. A 100-foot tape and scales 1, 2, and 3 feet long with large numbers suitable for photo documentation will be required. A diver's compass and accurate depth gauge, as well as survey buoys, will also be required.

The effectiveness of cathodic protection systems is measured as follows:

- On mooring buoys - either an underwater voltmeter as shown in Figure 3-23 or a portable voltmeter and portable reference electrode as shown in Figure 3-22 can be used.
- On chain - an underwater voltmeter is used.

To record any findings underwater, a grease pencil and Plexiglas slate are required. An underwater camera with extra film is required for photographic documentation and a video recording system may be required. An inclinometer is required for obtaining the angles of mooring chains in nonriser-type moorings and spread moorings. Marker tags are used to relocate or mark links or accessories. Transits and targets are required for locating buoy positions. Because UCT divers need high mobility, and because of the depth of water in which they will be working, the cleaning operations to be performed for inspection work generally require only hand tools, such as wire brushes and scrapers.

At times ROVs may be used to supplement mooring inspections (see Section 2.9).

## 3.6 CONCRETE STRUCTURES

### 3.6.1 Types of Marine Concrete Structures

Concrete is widely used in the marine environment as a construction material because of its many desirable properties. In its plastic state, concrete is easily mixed, handled, transported, and placed into forms. The strength of concrete can be regulated by adjusting the quantities of cement, aggregate, water, admixtures, and, in particular, the water-to-cement ratio. This ratio is one of the prime considerations in concrete mix design not only to provide adequate concrete strength, but, equally important, to provide long term durability of concrete in the harsh marine environment.

The performance of a concrete structure is most affected by the care taken in its construction and installation. Properly made concrete is highly durable in marine applications, exhibiting resistance to corrosion of reinforcing steel, chemical deterioration, weathering, erosion, and structural damage. Concrete is relatively strong under compressive loading, and with steel reinforcing resists bending and tensile forces. Concrete can be cast in place at the job site, precast into the required shape at a concrete plant and shipped to the site, or prestressed before installation to accept additional loading. With proper procedures (see Section 2.10), concrete can be rapidly placed underwater where it will harden into good quality concrete.

Circular or square concrete piles (Figure 3-6) are widely used to support piers, wharves, and other structures. Concrete is used as a decking material for many waterfront facilities and in retaining wall structures, such as those needed for closed piers and wharves, bulkheads, quaywalls, dry-docks, and seawalls. It is also used in pavements,

bridge foundations, boat loadings and ramps, breakwaters, undersea cable and pipeline stabilization, and offshore structures.

### 3.6.2 Deterioration of Marine Concrete Structures

The most common damage resulting from the premature deterioration of concrete structures in or near seawater is cracking and loss of material (or cross section). Softening of the concrete due to chemical action is another form of damage but less common than cracking. As shown in Table 3-5, the damage to concrete is generally most severe in the splash and tidal zones, but does occur in all zones. The different exposure zones are shown in Figure 3-11.

Deterioration of concrete waterfront structures is caused primarily by:

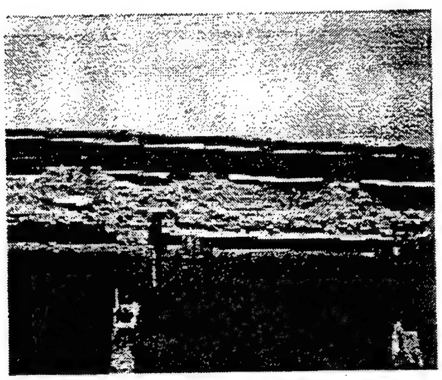
- Corrosion of steel reinforcement
- Repetitive freezing and thawing of moist concrete
- Abrasion
- Chemical deterioration
- Structural overloading

The three most common visual signs of concrete deterioration in marine structures are: cracking, disintegration, and spalling, as illustrated in Figures 3-43 and 3-44. Disintegration is defined as an overall decay of the concrete involving loss of strength of the cement and sand paste and subsequent loosening or loss of coarse aggregate particles. Spalling is defined as a localized area or fragment of concrete falling away from the structure. Both disintegration and spalling can expose reinforcing steel.

Table 3-5. Types of Damage in Marine Concrete

| Description of Damage | Zone (Location)[a] | | Common Causes of Damage | | | | | |
|---|---|---|---|---|---|---|---|---|
| | Observed | Most Severe | Corrosion of Reinforcement | Freeze-Thaw | Abrasion | Sulfate Attack | Chemical Reaction of Aggregates | Structural Overload |
| Cracking | All | T | X[b] | X | | | X | X |
| Loss of Material-[c] Exposed Reinforcement and/or Aggregate | All | S, T | X | X | X | X | X | X |
| Material | S, T, Su | S, T | | | | X | | |

[a]A = Atmospheric zone; S = Splash zone; T = Tidal zone; Su = Submerged Zone; M = Mud zone (see Figure 3-11).
[b]Rust stains on the concrete surface are generally a symptom of corrosion of the reinforcement.
[c]Loss of material from spalling, scaling, disintegration.



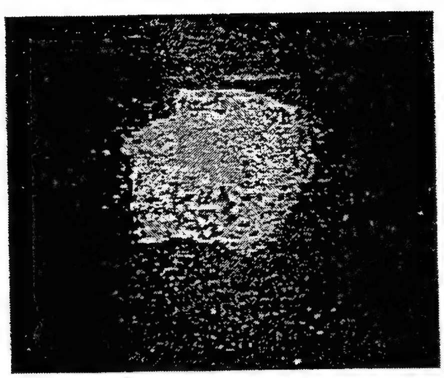


Figure 3-43.
Concrete deterioration.

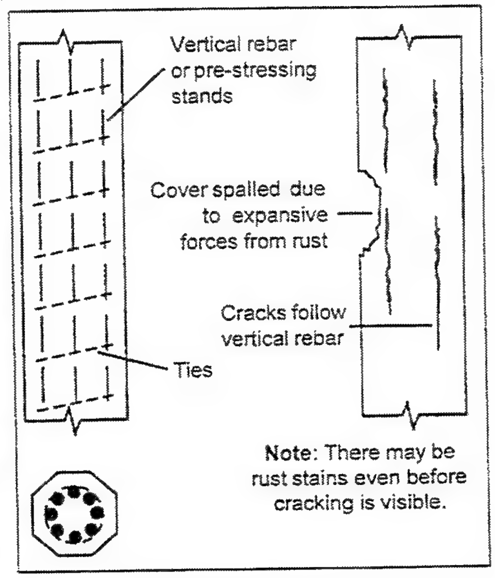



Figure 3-44
Examples of damage resulting from corrosion of reinforcement.

The causes for each symptom of deterioration are many and varied, and in most cases of progressing deterioration, they occur simultaneously. Much concrete deterioration in the marine environment starts as a result of poor construction techniques and inadequate inspection and quality control during construction. To develop a suitable and adequate concrete repair procedure, the cause of deterioration must be determined. Causes of concrete deterioration are described.

### 3.6.2.1 Corrosion of Reinforcing Steel

With the exception of mass gravity structures, most marine concrete structures use steel reinforcement. This reinforcement, to be most effective, is nearly always located within a few inches of the concrete surface, making the steel susceptible to corrosion if it does not have adequate cover of good quality concrete. Corrosion is more likely to occur if the concrete is overly porous or if cracking is initiated by some action.

The reinforcing steel corrosion products (rust) can increase the volume of the rusted area up to eight times. This leads to cracking of the concrete cover in lines parallel to the reinforcing steel. Eventually spalling results, and in cases of close reinforcement spacing, a complete delamination of the concrete surface can occur as illustrated in Figure 3-44.

All concrete is porous to some extent. The degree of porosity is dependent primarily on the water/cement (w/c) ratio of the concrete mix and on good construction practices. The lower the w/c ratio, the more dense (less porous) the concrete which limits the rate at which water, dissolved oxygen, and chloride ions reach the reinforcing steel, and lengthens the time for corrosion of the rebar to damage the concrete.

For example, reinforced marine concrete made with a w/c ratio of 0.6 to 0.7 (7 or 8 gallons of water per 94-pound sack of cement) will show rebar corrosion, cracking, and spalling in a very few years, whereas well made concrete with a w/c of 0.4 (about 4-1/2 gallons of water per sack of cement) will likely serve several decades or more before serious deterioration occurs.

#### 3.6.2.2 Freeze/Thaw Deterioration

Freeze/thaw deterioration is the freezing of absorbed moisture or water in porous concrete exposed to subfreezing temperatures. This is one of the most common causes of concrete deterioration in the tidal and splash zones. Upon freezing, this entrapped water expands and cracks the concrete. Upon thawing, the cracked surface disintegrates. Repeated cycles of freezing and thawing can lead to partial or even total loss of the concrete cross section, thus exposing the reinforcing steel which then rapidly corrodes as illustrated in Figure 3-45. The best prevention of freeze/thaw damage is to use air-entrained concrete with a rich cement content and a low water/cement ratio.

Precast concrete piles may have a cast-in-place jet pipe that was not filled with concrete after the pile was driven. When the water in the pipe freezes, it can cause longitudinal cracks in the pile, as illustrated in Figure 3-46.

#### 3.6.2.3 Abrasion Wear

Abrasion is defined as erosion of a concrete surface by the physical action (impact and rubbing) of external loadings or abrading agents. Deck slabs are subject to abrasion by vehicular traffic and loading equipment. Deck edges and wharf faces at berthing spaces without adequate fendering are abraded by moored vessels. Frequently, concrete piles and walls are abraded in the tidal zone by floating debris and ice moved by currents, waves, propeller wash, and tide changes. Less frequently, submerged concrete, especially at the mudline, is abraded by silt, sand, and debris churned up by moving water. Figure 3-47 illustrates the effects of abrasion on a concrete pile.

#### 3.6.2.4 Chemical Deterioration

The most significant and serious saltwater chemical reaction to hardened concrete is the combining of sulfates in seawater with chemicals in the cement paste, referred to as sulphate attack. This reaction can produce internal expansion and cause cracking. More

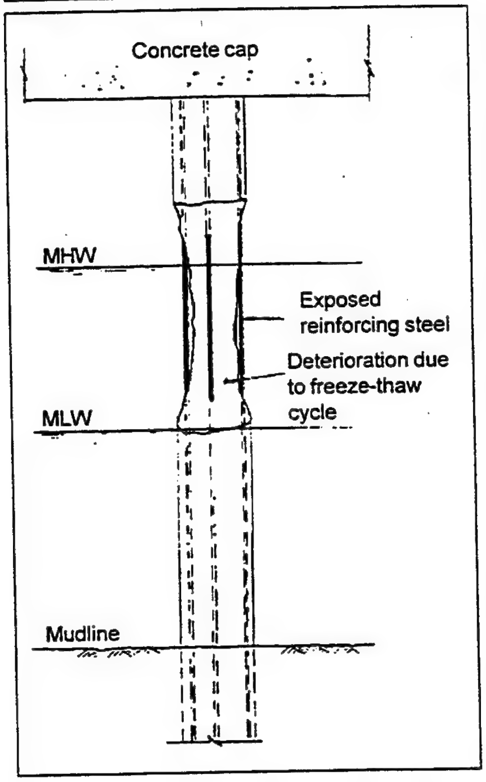



Figure 3-45. Freeze/thaw deterioration.

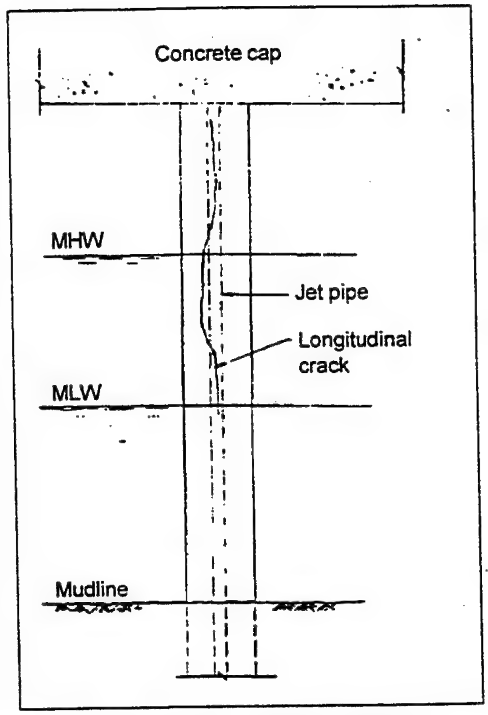



Figure 3-46. Internal freezing of jetted-in concrete pile.

commonly, however, the hydrate cement paste looses strength and becomes soft. Aggregate particles become exposed or fall from the concrete mass because of the weak cement paste.

### 3.6.2.5 Axial Overloading

Deterioration of concrete piles from axial overloading can be a cause of eventual failure of the pile. Overloading can result from superimposed "dead" and "live" loads exceeding the bearing capacity of the pile, and also from overstressing at the time of pile driving. Pile driving overloading often results in hairline cracks at the top of the pile or circumferential cracks at other locations along the pile that are difficult to

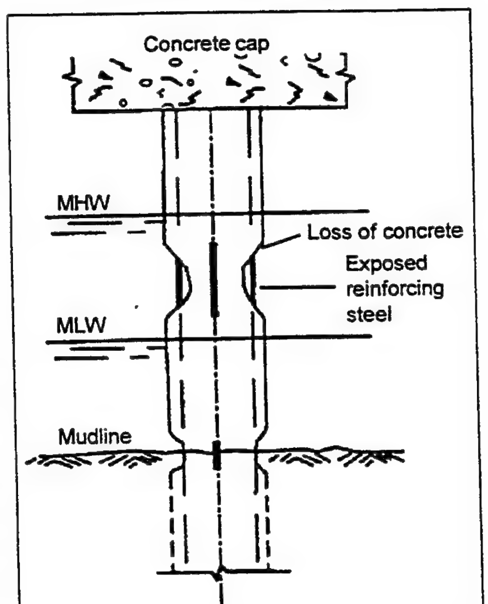



Figure 3-47. Abrasion of concrete pile.

see, as illustrated in Figure 3-48. As marine growth covers the pile, the cracks become extremely difficult to detect.

### 3.6.2.6 Shrinkage

Shrinkage or contraction can occur from moisture or temperature changes. Hardened concrete that looses internal water due to evaporation will shrink. Any temperature decrease of the concrete will cause contraction. The major cause of microcracks within concrete is from high temperatures generated from the normal hydration of cement. The concrete hardens at a high temperature and later cools to ambient temperatures. Precast concrete members that have been steam cured are particularly susceptible to microcrack formation. If the shrinkage or contraction is restrained, internal stresses may develop in sufficient magnitude to cause significant cracks in the structure.

Variations in atmospheric temperature cause a change in temperature of a hardened concrete mass, which results in volumetric changes. Provisions must be made to permit this expansion and contraction process to take place. Failure to do so will result in contraction stresses (tension), which may cause cracking, or expansion stresses (compression), which may lead to spalling.

### 3.6.2.7 Swelling

Concrete that increases in moisture content by absorbing water or increases in temperature will swell or expand. Typically, swelling by water absorption is not a concern unless precast dry-concrete members are used. Temperature increases from daily and seasonal changes may cause cracking in some concrete members.

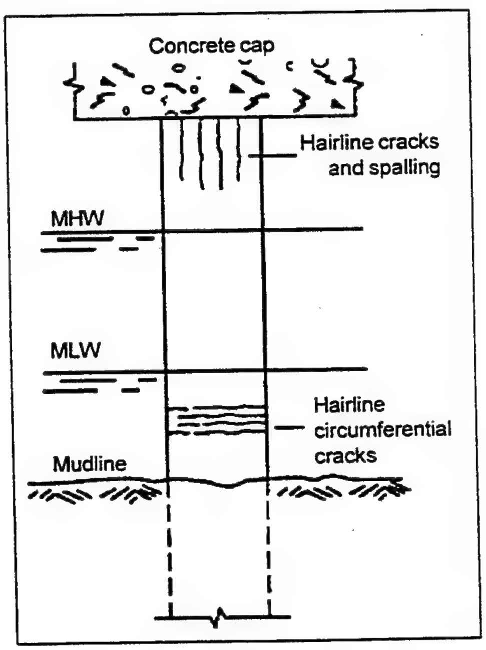


Figure 3-48. Overloading of concrete pile during pile driving.

### 3.6.2.8 Other Deterioration Factors

The preceding has discussed deterioration in concrete caused by improper selection or proportioning of concrete materials, faulty construction methods and procedures, and attack by environmental forces. Of equal importance, and a major cause of much concrete deterioration, is poor design of concrete structural details.

A few examples of poor design and construction details that contribute to concrete failure and deterioration are:

- Congestion of reinforcing steel
- Lack of adequate cover for reinforcing steel
- Abrupt change in size of section

- Re-entrant corners
- Lack of chamfers and fillets at corners
- Rigid joints between precast units
- Construction joint leakage
- Poorly designed scuppers, drips, and curb slots
- Inadequate drainage
- Too little gap at expansion joints
- Incompatibility of materials or sections

### 3.6.3 Concrete Inspection Procedure

#### 3.6.3.1 Visual Inspection

Levels I and II visual inspection of concrete waterfront structures should proceed as shown in Table 3-6.

#### 3.6.3.2 Level III Nondestructive Inspection of Concrete

The qualitative data obtained from visual inspections are sometimes inadequate to accurately assess the condition of the structure. In these instances, quantitative data obtained from nondestructive testing instruments can assist the facilities engineer in determining the condition of the structure. Three specialized instruments have been developed for underwater inspection of concrete structures. These instruments are the:

- Magnetic rebar locator - used to determine the location and orientation of rebar in concrete structures and to measure the amount of concrete cover over the rebar.
- Rebound hammer - used to evaluate the surface hardness of the concrete and obtain a general condition assessment.
- Ultrasonic system - used to obtain a general condition rating and indication of overall strength of the concrete based on sound velocity measurements through a large volume of the structural element.

Each instrument consists of an underwater sensor connected to a topside deck unit through an umbilical cable. The deck unit contains the signal conditioning electronics and data acquisition system. To operate the instruments, the diver has to position the underwater sensor on a previously cleaned portion of the structure surface and a person topside must operate the data acquisition system in order to collect and store the data. Each instrument is independently operated and provides unique information to help assess the condition of the concrete structure.

### 3.6.4 Equipment and Tools Required

To perform a thorough inspection, the marine growth on the structure must be removed. A "Barnacle Buster" or pneumatic chipping gun is an efficient method of removing marine growth from concrete surfaces. Various types of high-pressure water jet cleaning systems are also effective. Exercise care in the use of these methods because

Table 3-6
Concrete Structure Underwater Inspection Checklist

| Checkpoint | Description |
|---|---|
| 1 | Inspect the structure beginning in the splash/tidal zone. This is where most mechanical and biological damage is normally found. |
| 2 | Clear a section about 18 to 24 inches in length of all marine growth. |
| 3 | Visually inspect this area for cracks, abraded surface spalling, or mechanical damage, and exposed reinforcing steel. |
| 4 | Sound the cleaned area with a hammer to detect any loose layers of concrete hollow spots in the pile, structure, or soft concrete. A sharp ringing noise indicates sound concrete. A soft surface will be detected, not only by a sound change, but also by a change in the rebound, or feel, of the hammer. A thud or hollow sound indicates a delaminated layer of concrete, most likely from corrosion of steel reinforcement. |
| 5 | Descend, visually inspecting the pile or structure where marine growth is minimal, and sound with a hammer. |
| 6 | Inspect in greater detail the base of mass structures, such as foundations, quaywalls, breakwaters, or bridge piers. These types of structures are prone to undermining by wave and current action, which, if not rectified, could lead to failure of the structure. |
| 7 | At the bottom, record the water depth along with any observations of damage on a Plexiglas slated. |
| 8 | After returning to the surface, immediately record all information into the inspection log.<br><br>NOTE: If signs of deteriorations are found, then a Level III inspection, involving either nondestructive or destructive tests, may be required. Refer to the Level III Test Procedures for Concrete Inspection for mechanical and electrical test methods. |
| | Exposed Area Under Pier or Along Wharf or Dolphin Assembly |
| 9 | Check pile caps and bearing, batter, and fender piles for damaged or broken members, cracks, and spalling of concrete, rust stains, and exposed reinforcing steel. |
| 10 | Sound the piling or structure with a hammer to detect any loose layers of concrete or hollow spots. A sharp ringing noise indicates sound concrete. A soft surface will be detected, not only by a sound change, but also by a change in the rebound, or feel, of the hammer. A thud or hollow sound indicates a delaminated layer of concrete, most likely from corrosion of steel reinforcement.<br><br>NOTE: If signs of deteriorations are found, then a Level III inspection, involving either nondestructive or destructive tests, may be required. Refer to the Level III Test Procedures for Concrete Inspection for mechanical and electrical test methods. |

they may further damage a deteriorating concrete structure. If minimal marine growth is found in the splash/tidal area, small hand tools, such as wire brushes and scrapers, are sufficient. Refer to Section 2.3 for information on equipment for removing marine growth. A hammer for sounding and an accurate water-depth gauge will be required. Record observations on a Plexiglas slate with a grease pencil. Use underwater video cameras for permanent visual documentation.

### 3.6.4.1 Magnetic Rebar Locator

The magnetic rebar locator shown in Figure 3-49 is based on a commercial instrument that detects the disturbances in a magnetic flux field caused by the presence of magnetic material. The magnitude of this disturbance is used to determine the location and orientation of rebar in concrete structures and to measure the amount of concrete cover over the rebar.

The system consists of an underwater test probe, an umbilical cable, and a topside data acquisition unit (DAU) including printer.

The test probe consists of two coils mounted on a U-shaped magnetic core. A magnetic field is produced in one coil and the disturbance-induced magnetic field in the rebar is measured in the other coil. The magnitude of the induced current is affected by both the diameter of the rebar and its distance from the coils. Therefore, if either of the parameters is known, the other can be determined.

By scanning with the probe until a peak reading is obtained, the location of the rebar can also be determined. A maximum deflection of the meter needle will occur when the axis of the probe poles are parallel to and directly over the axis of a reinforcing bar, thus indicating orientation.

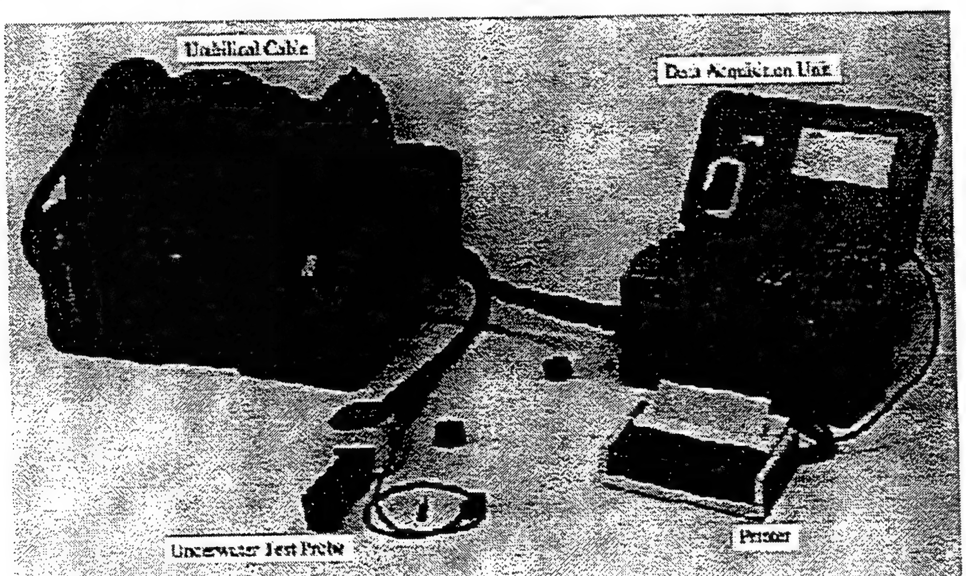



Figure 3-49. Underwater rebar locator system.

The underwater rebar locator is calibrated for rebar that varies from No. 3 to No. 16 in size. The meter can be used to measure the depth of concrete cover over rebar in the range of 1/4 to 8 inches thick, or conversely, it can measure the diameter of the rebar. The best accuracy ($\pm$10 percent) is obtained for concrete cover less than 4 inches thick.

- System Limitations. The presence of other metallic objects in the vicinity where the measurements are being made can affect the operation of the rebar locator. For example, in heavily reinforced structures, the effect of nearby rebar cannot be eliminated and accurate depth readings are difficult or impossible.

If the separation of two parallel rebars is at least three times the thickness of the concrete cover, this effect can be neglected.

The presence of rebar perpendicular to the axis of the underwater probe has less effect on the measurement of concrete cover than that of parallel rebar, and in most instances it can be ignored.

### 3.6.4.2 Rebound Hammer

The underwater rebound hammer system, shown in Figure 3-50, is a surface hardness tester which can be used to obtain a general condition assessment of concrete. The system consists of an underwater rebound hammer, an umbilical cable, and a topside data acquisition unit (DAU) including printer. The rebound hammer is mounted in a waterproof housing which contains an electrical pickup to sense the position of the rebound mechanism. The umbilical connects the underwater rebound hammer to DAU that contains the signal conditioning electronics and data acquisition system.

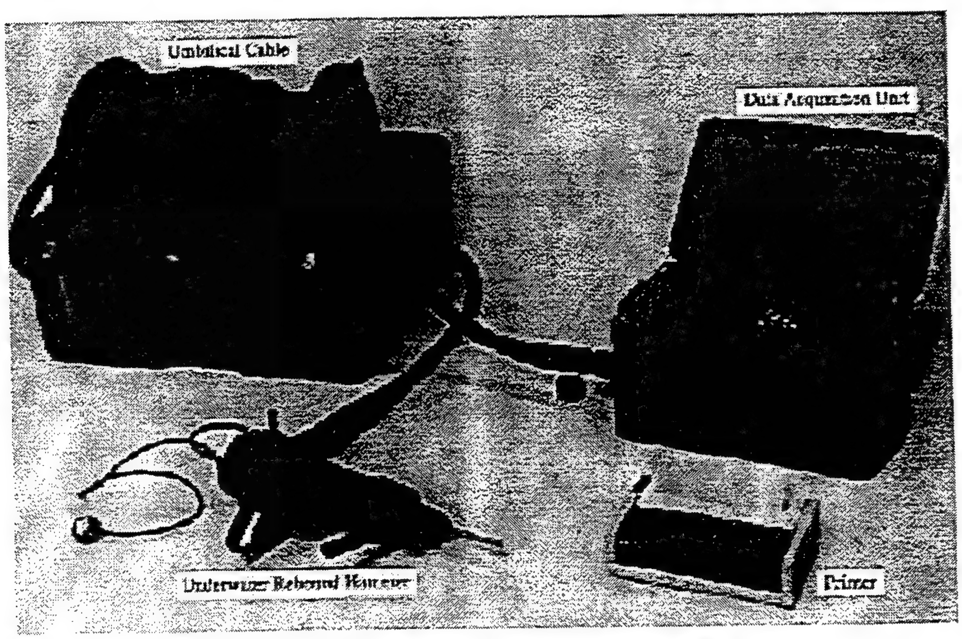



Figure 3-50. Underwater rebound hammer system.

The rebound hammer correlates the rebound height of a spring-driven mass after it impacts the surface of the concrete with the compressive strength of the concrete under test. The spring-driven mass slides on a guide rod within the tubular housing. When the impact plunger is pressed firmly against the concrete surface, a trigger releases the spring-loaded mass causing it to impact the plunger and transfers the energy to the concrete surface. The mass then rebounds and the rebound height is correlated to the surface hardness of the concrete.

> WARNING
>
> Do not operate the rebound hammer with the impact plunger in contact with human body parts; serious injury can result.

A general calibration chart that relates the rebound number to cube compressive strength for the underwater rebound hammer is shown in Figure 3-51.

The pressure housing has a depth rating of 190 feet and it is pressure compensated at 5 psi over the ambient pressure. Air is supplied to the rebound hammer from a scuba tank through the umbilical cable via an external pressure regulator to maintain the positive pressure differential inside the housing.

- System Limitations. The following characteristics of concrete can affect the correlation of the rebound number with the actual surface hardness and should be understood before using the instrument:

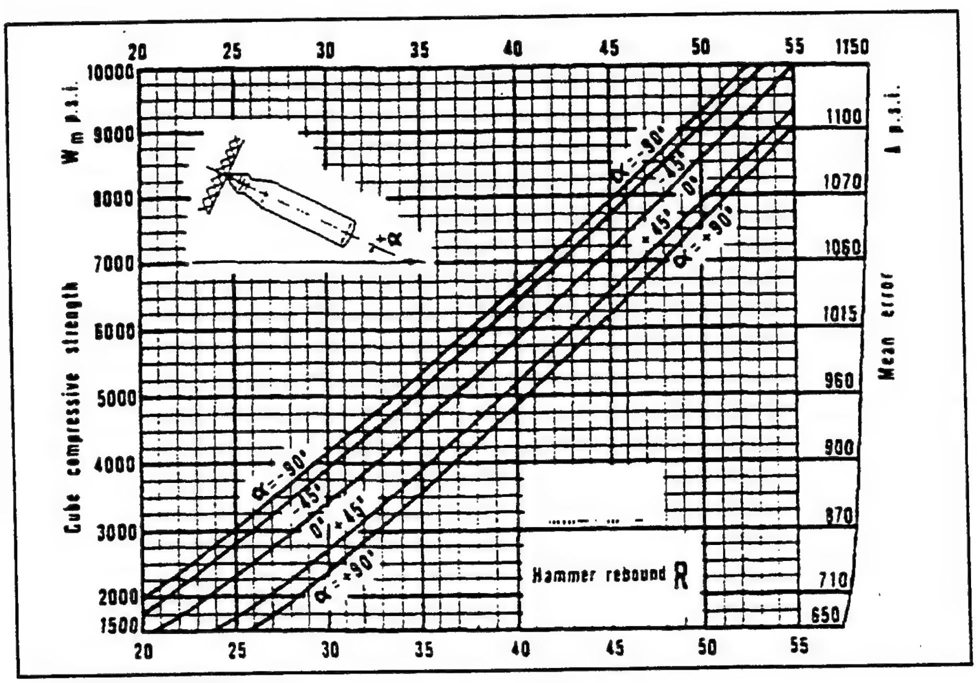



Figure 3-51. H-Meter calibration chart.

1. Higher rebound numbers are generally obtained from smoother surfaces and the scatter in the data tends to be less. Minimizing the data scatter increases the confidence in the test results. Therefore, underwater concrete surfaces must be thoroughly cleaned and smoothed with a carborundum stone (or similar abrasive) before measurements are taken.

2. Water-saturated concrete tends to show rebound readings approximately 5 points lower than for the same concrete tested dry. This affects the comparison of data taken above and below the waterline.

3. Type of aggregate and cement affects the correlation of the rebound numbers with actual compressive strength of the concrete under test. A calibration curve is required for each particular concrete mix to assure accuracy. Since this is not practical for most situations, the data should only be used for making comparative measurements from one location to another within a uniform concrete structure.

Because of these limitations, the estimation of concrete compressive strength obtained with a rebound hammer is only accurate to about ±25 percent. This applies to concrete specimens cast, cured, and tested under the identical conditions as those from which the calibration curves were established.

The rebound hammer is primarily useful for checking surface compressive strength or surface hardness and uniformity of concrete within a structure. It can also be used to compare one concrete structure against another if they are known to be reasonably similar.

### 3.6.4.3 Ultrasonic System

The ultrasonic system shown in Figure 3-52 is used to obtain a general condition rating and indication of overall strength of the concrete based on sound velocity measurements through a large volume of the structural element. It is recommended that the underwater ultrasonic system be used primarily for checking the uniformity of concrete from one test location to another in a given structure. If the data consistently indicate poor or very poor quality concrete, core samples must be taken and standard compression tests performed to confirm the results.

The system consists of two different underwater transducer holders for direct and indirect sound velocity measurements. An umbilical cable connects either the direct or indirect transducer holder to the topside DAU. The DAU contains most of the signal conditioning electronics and data acquisition system.

Ultrasonic techniques use the transit time of high-frequency sound waves through concrete to assess its condition. Ultrasonic testing procedures for concrete have been standardized by ASTM C597 and test equipment is available from commercial sources for in-air testing. Measuring sound velocity in concrete requires using a separate transmit and receive transducer to avoid energy scattering and reflection problems. Sound velocity is calculated by measuring the time required to transmit over a known path length. The average sound velocity obtained should only be used as an indicator of concrete quality and not as a measurement of compressive strength. Table 3-7 presents some suggested condition ratings for concrete based on sound velocity measurements.

The two methods used to measure sound velocity in concrete are direct and indirect. The most preferred method is direct transmission where the transducers are positioned on opposite sides of the test specimen and the waves propagate directly toward the receiver. This method provides maximum sensitivity with a well defined path length.

Indirect transmission is used when only one surface of the concrete is accessible, such as a concrete retaining wall: both transducers are placed on the same side of the concrete. With this method, energy scattered by discontinuities within the concrete is detected by the receive transducer.

Table 3-7
General Condition Rating Based on Sound Velocity

| Condition Rating | Sound Velocity (ft/sec) |
|---|---|
| Excellent | > 15,000 |
| Good | 12,000 - 15,000 |
| Questionable | 10,000 - 12,000 |
| Poor | 7,000 - 10,000 |
| Very Poor | < 7,000 |

- Transducer Holders. Two types of transducer holders are provided with the ultrasonic system. The direct transducer holder (Figure 3-53) is used to examine structures with accessible opposing surfaces; for example, concrete piles. The indirect transducer holder (Figure 3-54) is used to examine structures with only one accessible surface; for example, concrete bulkheads.

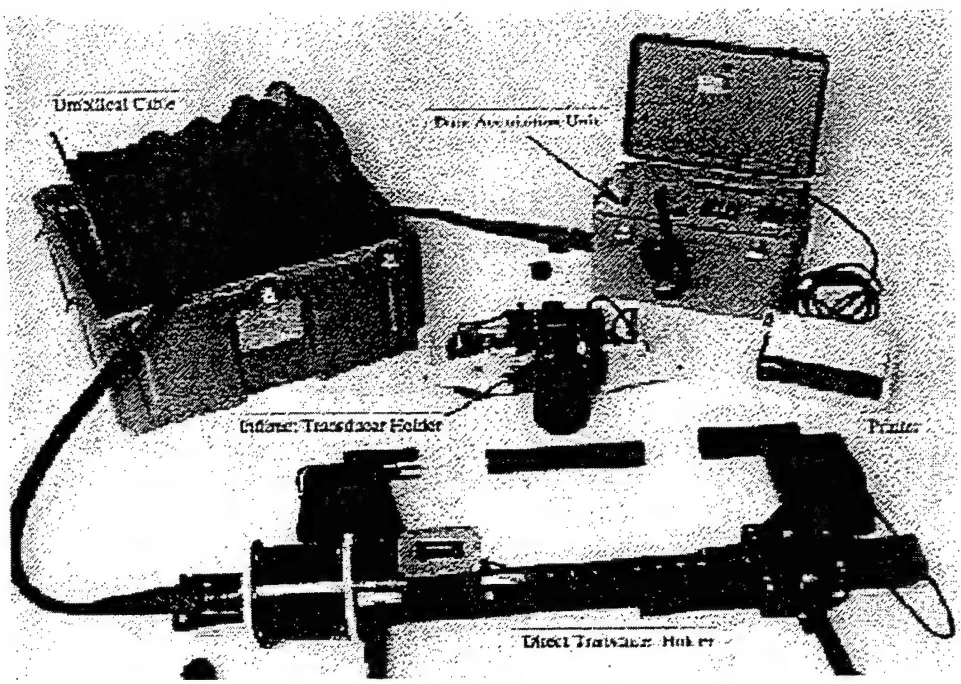


Figure 3-52. Underwater ultrasonic test system.

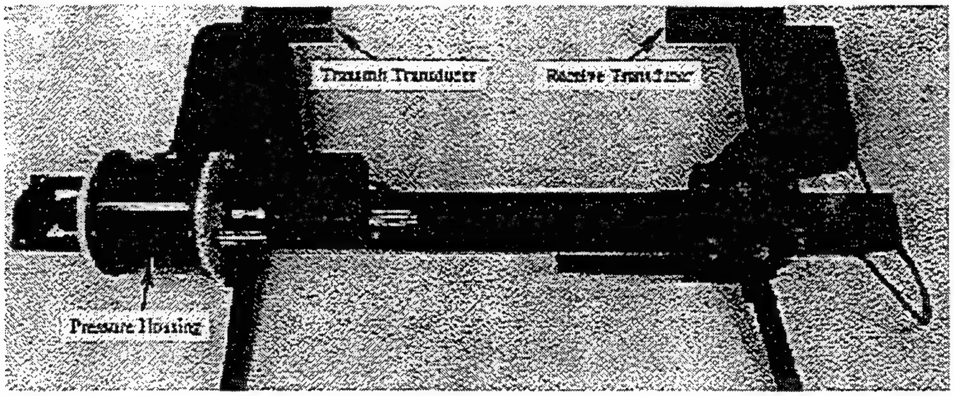


Figure 3-53
Director transducer holder.

The direct transducer holder framework can be adjusted to accommodate concrete pile sections that range from 8 inches to 24 inches thick. The digital display of sound wave transit time provides feedback to help the diver position the transducer holder for optimum results.

The indirect transducer holder is very similar to the direct transducer holder in operation except for the path length measurement which is fixed at 12 inches. A suction cup was added to the indirect holder to force the transmit transducer firmly against the concrete surface under test and provide a reaction force for the diver. A small suction pump is used to pump water from the cup to provide a holding force of about 25 pounds depending on the surface condition of the concrete.

- System Limitations. Results obtained with the ultrasonic test system are affected by the following factors which influence the quality of the data:

1. Concrete Surface Finish - The smoothness of the surface under test is important for maintaining good acoustical coupling between the transmit transducer and the surface of the concrete. A coupling agent, such as silicone grease, must be placed between the transmit transducer and the concrete surface to transfer maximum energy. If a coupling agent is not used, the transmitted signal will be severely attenuated which results in large errors in the measurement of the transit time.

2. Reinforcing Steel - Sound velocity measurements taken near steel reinforcing bars may be higher because the sound velocity in steel is from 1.2 to 1.9 times the velocity in concrete. The effect is small when the axis of the rebar is perpendicular to the direction of sound propagation and the correction factors are on the order of 1 to 4 percent depending on the quality of the concrete. If the axes of the rebar are parallel to the direction of sound propagation, reliable corrections are difficult. Therefore, it is recommended that sound transmission paths be chosen that avoid the influence of the rebar.

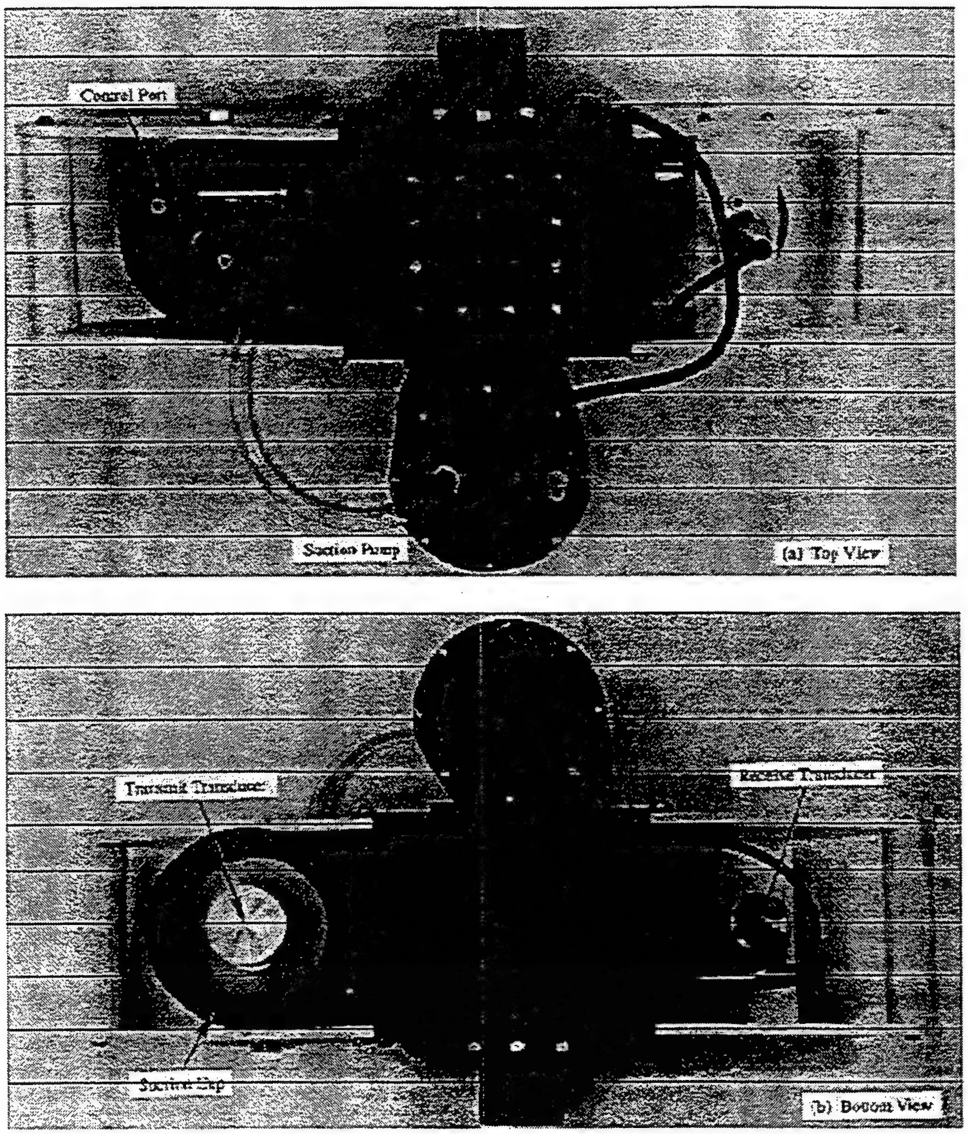



Figure 3-54. Indirect transducer holder.

3. Signal Detection Threshold - The signal detection threshold of the ultrasonic system can cause erroneous transit time data to be recorded. This happens when the amplitude of the first peak of the received signal is below the threshold triggering level of the system. When the instrument detects a following peak, this causes an apparent transit time increase of one-half wavelength or more.

- Begin the inspection at the waterline, checking for excessive weathering and abrasion deterioration, and loss of mortar from the joints.
- Inspect below the waterline, taking note of the general condition of the wall, and paying particular attention to the joints between each stone.
- If there are significant gaps between stones or stones are missing, note the location, depth, and length of missing stone.
- Continue to the bottom of the structure and note any undermining or scouring of the material under the wall structure.
- At any missing stone or undermining, probe the cavity to estimate the extent of the void (if any) behind or below the wall.
- Record the depth of the water at the base of the wall.
- After returning to the surface, immediately transcribe all information into the inspection log if information has not been communicated via hardwire. Also, record in the log the general condition of the wall above the waterline, especially noting all joints from which mortar has washed out.

### 3.8.4 Equipment and Tools Required

Since the underwater inspection of stone masonry structures involves only a cursory inspection of the joints between stones and the general condition of the wall and its foundation, only a few tools are required. A ruler is used to determine the width and depth of cracks and open joints, as well as the size of missing stones or pieces of stone. It is also useful for quantifying the amount of scouring that has occurred. A length of small-diameter rebar or other suitable probe can be used to check for voids in the fill behind or below the wall. A Plexiglas slate and a grease pencil are used underwater to record any pertinent information, or the information is communicated to topside personnel via hardwire. Small hand tools, such as wire brushes and scrapers, are also useful to clear off cracks and joints.

## 3.9 COASTAL PROTECTION STRUCTURES

Structures designed to reduce the erosive effects of wave action, or to protect harbors from excessive wave action and the formation of sandbars, are classified protection structures. The common coastal protection structures are seawalls, groins, jetties, and breakwaters. NAVFAC Mil-Hdbk-1025/4, "Seawalls, Bulkheads, and Quaywalls" and NAVFAC DM-26.02, "Coastal Protection," provide additional information on the design and configuration of coastal protection structures.

### 3.9.1 Seawalls

Seawalls are massive coastal structures built along the shoreline to protect coastal areas from erosion caused by waves and flooding during heavy seas. Seawalls are con-

structed of a variety of materials including rubble-mounds, granite masonry, or reinforced concrete elements (Figure 3-64). They are usually supplemented by steel or concrete sheet pile driven into the soil and are strengthened by wales and brace-type piles. Figure 3-65 shows three seawall configurations.

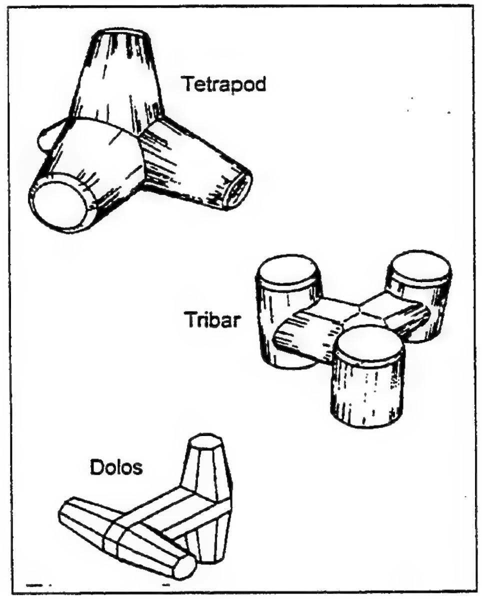



Figure 3-64. Precast concrete elements.

3.9.2 Groins

Groins are structures designed to control the rate of shifting sand by influencing offshore currents and waves so that erosion of the shoreline is minimized. Groins project outward, perpendicular to the shoreline, and are constructed of large rocks, precast concrete units, reinforced or prestressed concrete piles, steel sheet piles, or timber cribbing filled with rock. Figure 3-10 shows an example of a groin.

3.9.3 Jetties

Jetties are structures which extend from the shore into deeper water to prevent the formation of sandbars and to direct and confine the flow of water due to currents and tides. These structures are normally located at the entrance to a harbor or a river estuary. Jetties are usually constructed of mounds of large rubble to a height several feet above the high tide mark.

3.9.4 Breakwaters

Breakwaters are large rubble-mound structures located outside of a harbor, anchorage, or coastline to protect the inner waters and shoreline from the effects of heavy seas. These barriers help to ensure safe mooring, operating, loading, or unloading of ships within the harbor. Breakwaters may be connected to the shore or detached from the shore. There are three general types of breakwaters, depending on the type of exposed face. The exposed face may be vertical, partly vertical, and partly inclined, or inclined. Figure 3-66 shows a section of a breakwater.

3.9.5 Rubble-Mound Structures

Rubble-mound structures (Figure 3-67) are constructed on the seabed by dumping stones of various sizes from scows and barges until the mound emerges a certain distance above mean sea level. The outer layers of the mound are covered with armor consisting

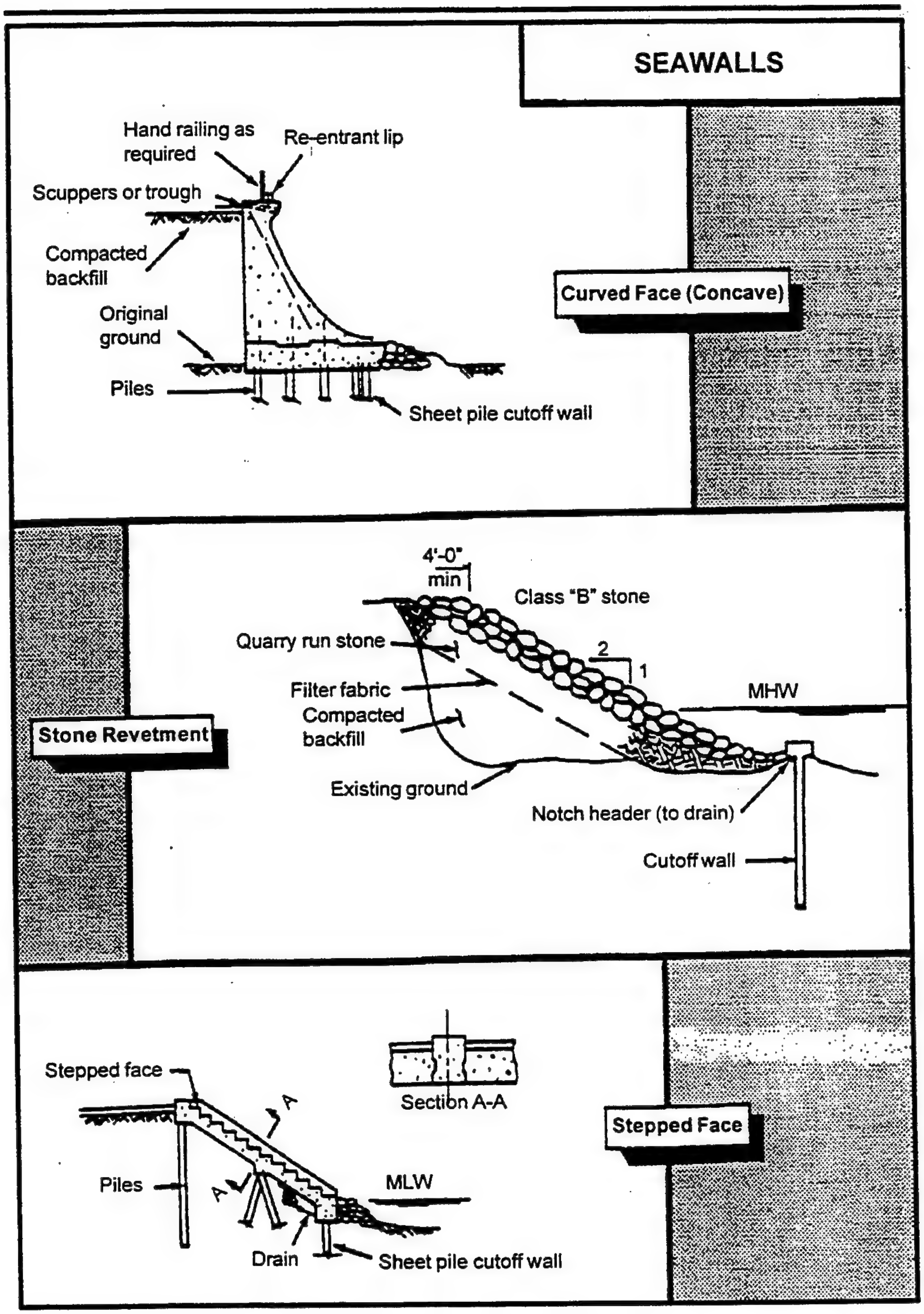



Figure 3-65. Seawall construction.

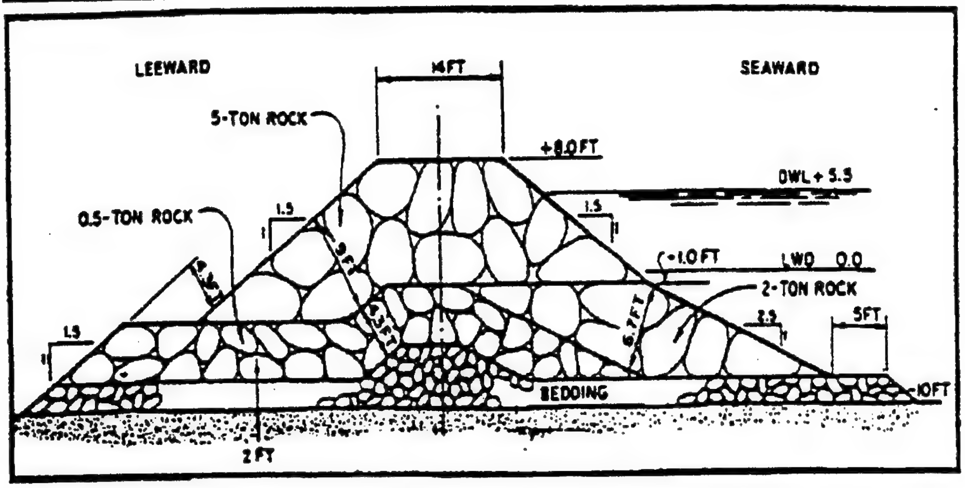



Figure 3-66. Typical breakwater section.

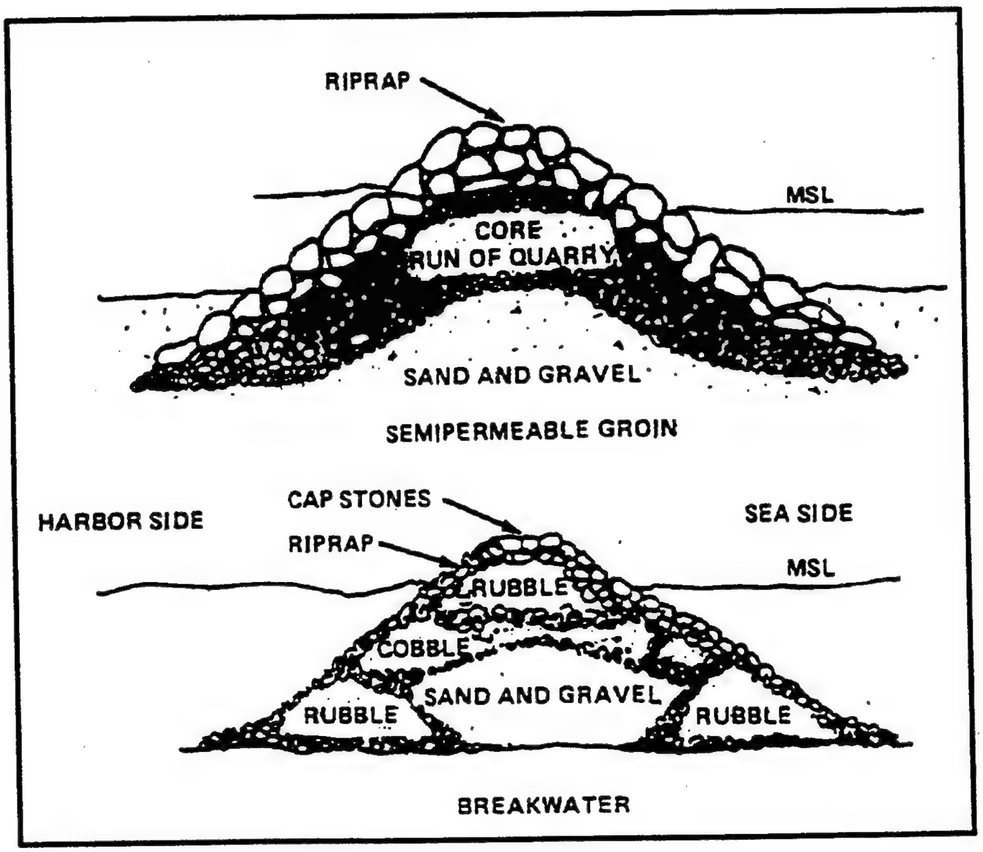



Figure 3-67. Typical rubble-mound structures.

either of large stone or precast concrete units of a number of possible shapes (Figure 3-64). Rubble is irregularly shaped rough stones, ranging in size up to 1,000 $ft^3$ each and weigh up to 90 tons each. Cobble, also used in rubble-mound structures, is rounded gravel or gravel fragments between 2-1/2 and 10 inches in diameter. Rubble-mound structures are used extensively, chiefly because they are adaptable to almost any depth of water in the vicinity of harbors and can be repaired readily.

### 3.9.6 Deterioration of Rubble-Mound Structures

The four principal types of deterioration in rubble-mound waterfront structures are:

- Sloughing of side slope
- Slippage of base material as a result of scour by currents
- Dislodgement of stones by wave action
- Excessive settlement of the seabed supporting the structure

During the inspection of seawalls, breakwaters, groins, and jetties, similar to those shown in Figure 3-67, the inspector should check for horizontal and vertical alignment. He should also be particularly watchful for signs of breakage or displacement of large stones or concrete armor elements, and washing out of substrate under the larger stones or concrete elements, particularly at the toe of the structure. These losses can be early signs of eminent structural failures if corrective action is not taken.

Other points to check include curbing, handrails, and catwalks (as applicable) for loose, missing, or broken sections, obstructions, and other hazardous conditions.

Inspection of rubble-mound structures should include:

- Erosion of core material by wave action
- Erosion of small stones in riprap
- Stability of armor stones or blocks
- Breakage and displacement of concrete armor elements
- Washing out of substrate at the toe of the structures
- Undermining of foundation
- High water mark; overtopping
- Settling of structures

### 3.9.7 Typical Inspection Procedure

Inspection of a rubble-mound structure should proceed as outlined in Table 3-9.

### 3.9.8 Equipment and Tools Required

Inspecting rubble-mound structures requires that divers be equipped with recording devices, such as a Plexiglas slate, grease pencil, and cameras.

Table 3-9
Rubble-Mound Structure Surface and Underwater Inspection Checklist

| Checkpoint | Description |
|---|---|
| 1 | Swim around the base of the structure looking for beginning weaknesses in the base, such as washout of small stones and core material. |
| 2 | Note signs of detrimental wave action, such as scouring and sloughing. |
| 3 | Record all pertinent information on a Plexiglas slate. After returning to the surface, transfer the information into the inspection log. |
| 4 | Record the result of the above-water inspection, include a description of the alignment and general condition of the mound, such as dislodgement of stones, gaps, and other weaknesses. |

## 3.10 GRAVING DRYDOCKS

### 3.10.1 Graving Drydock General Arrangement

Graving drydocks are waterfront structures that provide ship drydocking to allow repair or overhaul of underwater portions of ships, barges, and other waterborne craft. Although some stone masonry drydocks exist, most drydocks are primarily concrete structures with a steel or concrete end closure (caisson) to allow dewatering of the drydock. Figures 3-68 and 3-69 show the general arrangement of a typical graving drydock and caisson assembly.

#### 3.10.1.1 Caisson Assembly

The drydock caisson provides a waterproof closure at the drydock opening to allow dewatering and flooding of the drydock. During normal operations, the caisson is moved into position at the entrance of the drydock and ballasted down by flooding internal chambers within the caisson. When positioned properly, the caisson seal makes contact with the drydock seat, and dewatering of the drydock commences. At this point, hydrostatic pressure pushes against the outside of the caisson and holds it in position against the seat. At the completion of drydock operations (ship overhaul, etc.), the reverse procedure is followed to flood the drydock and move the caisson from the drydock entrance. As the caisson is moved into and out of position during docking operations, damage to the caisson assembly may occur due to impact with the surrounding structure. Also, because the caisson is usually left in position at the drydock entrance for several months, exposure to floating debris and/or small craft may result in impact damage on the outboard side of the caisson.

The caisson seal (either wood/rubber or all rubber) located around the perimeter of the caisson makes contact with the drydock seat to provide the "seal" between the caisson and drydock (refer to Figure 3-70). Either steel or MONEL mounting studs/nuts/washers are used to attach the seal assembly to the caisson.

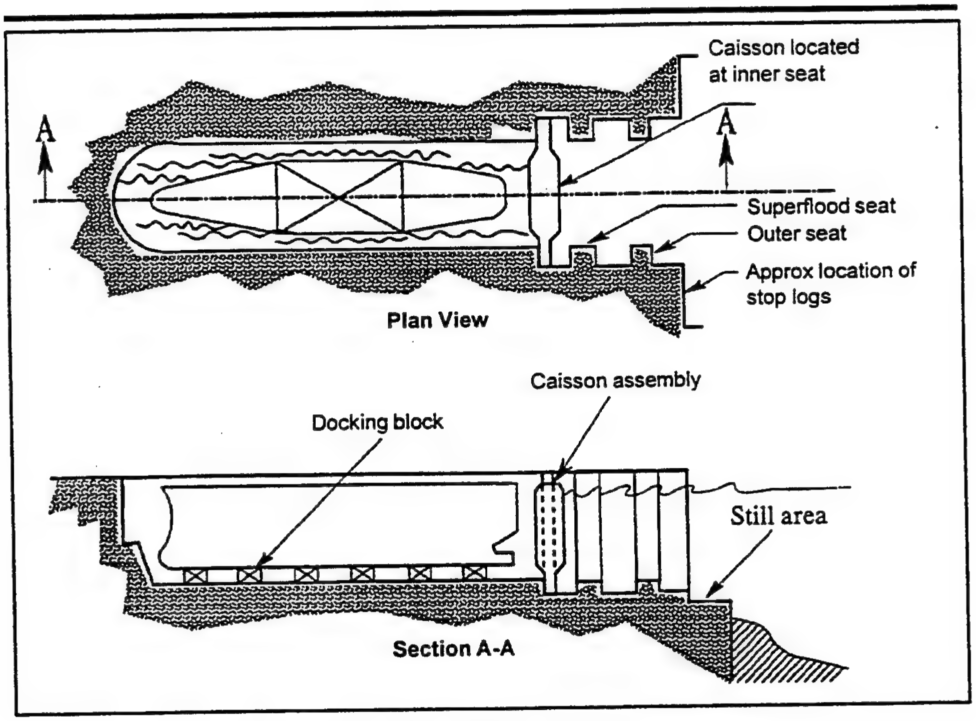



Figure 3-68. Graving drydock general arrangement.

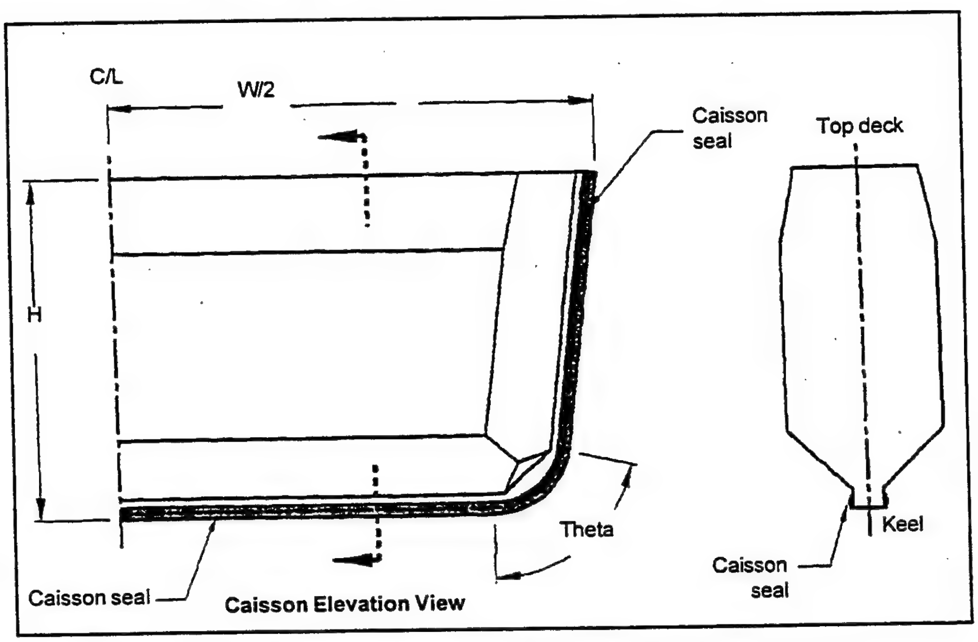



Figure 3-69. Drydock caisson general arrangement and seal location.

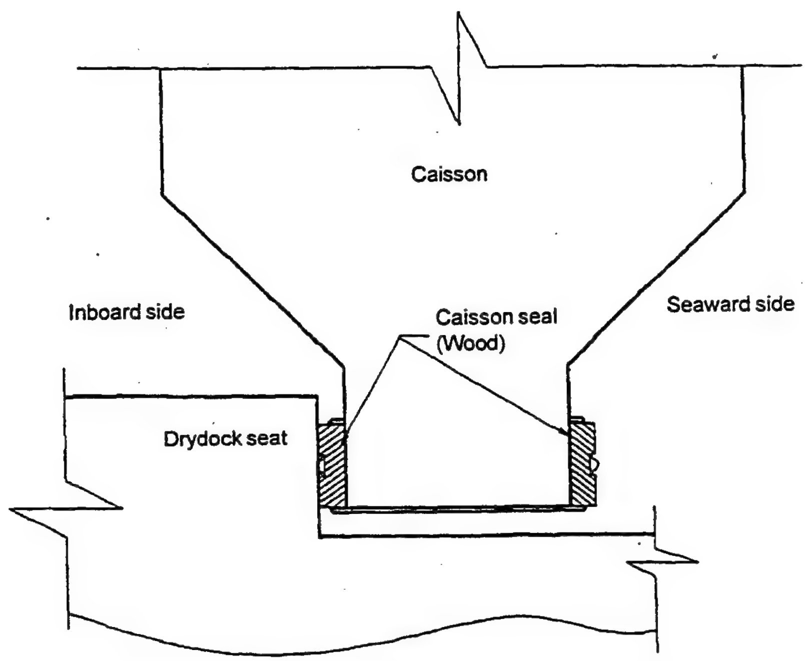



Figure 3-70. Plan view of drydock seat and caisson seal interface.

Drydock caissons are usually symmetrical about their longitudinal and transverse axes and have identical seal assemblies on two sides. This allows for reversing the caisson's inboard and outboard sides between docking operations. This also allows for a "dry" inspection of one side (inboard side of the caisson structure, but not the seal assembly) and alternates/minimizes "wet" exposure of the caisson structure and seal assembly between caisson overhaul.

### 3.10.1.2 Drydock Seat

As mentioned previously, the caisson seal assembly rests against the drydock seat to form the seal between the caisson and drydock. The surface of the drydock seat may consist of either a concrete surface, stone, or steel plating. In some cases, a drydock may have both an inner and outer seat. This configuration allows for two caisson positions at the drydock entrance area. A "dry" inspection of the inner seat can be accomplished when the caisson is positioned at the outer seat.

Some drydocks have the capability to "superflood." This is a condition where the water level inside the drydock is raised to a level higher than the level outside the drydock. When this occurs, the caisson shifts position and the outboard caisson seal rests against an opposing (inward facing) drydock seat.

3.10.1.3 Stop Logs

Some drydock configurations flood the drydock through flooding sluice gates and tunnels. These tunnels provide a path for water into the drydock during pumping/flooding operations. The underwater intake to the flooding tunnel is usually located just outboard of the caisson assembly. A steel plate called a "stop log" is normally installed across the intake opening and provides a secondary seal for the sluice gates inside the flooding tunnel. It is sometimes necessary for divers to inspect the stop logs and correct excessive leakage past the stop log seals prior to maintenance on valves and pumps located inside the flooding tunnel.

3.10.1.4 Drydock Sill

The drydock sill is located at the outboard edge of the drydock concrete structure at the seafloor/drydock interface. The drydock structure usually extends 5 to 10 feet above the seafloor at this point. Mud/silt buildup is a common problem in this area and must be removed if it extends above the drydock sill level. Otherwise, interference at the seal area and/or difficulty in positioning the caisson may result.

3.10.1.5 Docking Blocks

Docking blocks are used to support the keels and hulls of ships, barges, and vessels during overhaul operations. They are positioned and fixed on the drydock floor prior to ship docking and flooding operations. However, during ship docking operations, docking blocks may be inadvertently moved out of position. If this condition is suspected, divers are used to verify docking block position and in some cases remove (float) the blocks for subsequent reinstallation after dewatering the drydock.

3.10.2 Deterioration of the Graving Drydock Structure

In general, the concrete (or stone masonry) portion of the drydock is subject to deterioration through cracking, spalling, swelling, and disintegration, not unlike other concrete or stone masonry structures (refer to Section 3.6, Concrete Structures and Section 3.8, Stone Masonry Structures). However, during drydocking operations various conditions exist where additional damage to the structure may occur - namely, during ship and caisson positioning operations. Contact between the ship and drydock structure may occur as a ship is moved into the drydock. Likewise, inadvertent contact may occur during positioning of the caisson. In either of these cases, damage to the drydock structure may occur as a result of impact.

3.10.3 Deterioration of the Caisson Assembly

The caisson, whether a concrete or steel structure, is also subject to damage as a result of drydock operations. Deterioration of the caisson may occur as a result of caisson positioning operations (impact and/or abrasion damage) or from long term exposure to seawater (corrosion, biofouling, marine organisms, and/or impact).

For a steel caisson, corrosion of the metal structure is always a concern. Although the caisson is painted and "zincs" replaced at regular intervals during overhaul periods (usually every 5 to 6 years), corrosion may be accelerated in areas where the steel has been exposed (scraped, impacted, etc.).

Damage to a concrete caisson may show up as cracking, loss of material, swelling of the concrete, chemical deterioration, and/or exposure and corrosion of the steel substructure. These conditions can be a result of impact, abrasion, wave action, poor construction methods/techniques, and long-term exposure to a seawater environment.

### 3.10.4 Caisson Seal Deterioration

The cross section of a DM-29 wood caisson seal is shown in Figure 3-71. The wood portion of the seal transfers the load (due to hydrostatic pressure) from the caisson to the drydock seat and provides the primary seal. The small rubber gasket provides a secondary seal. Several physical and environmental conditions exist that can cause either accelerated or eventual deterioration of the wood:

- Splintering
- Fungi
- Rot
- Marine borers
- Insect infestation

Of these, splintering and marine borers are the most prevelant.

Similar to the caisson structure, the seal assembly is also subject to impact and/or abrasion damage during caisson positioning operations. Wood deterioration can be confirmed by visual inspection and is evidenced by wood splintering and cracking where impact has occurred.

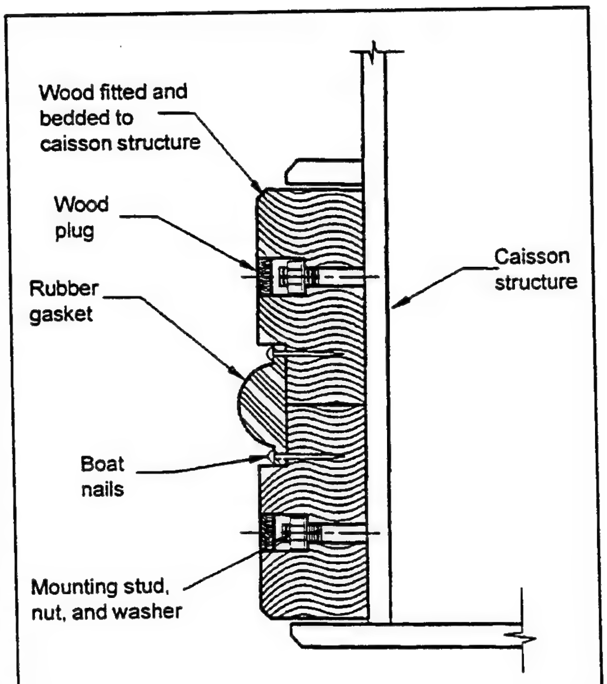



Figure 3-71. Typical caisson wood seal assembly.

Several marine organisms attack and infest wood structures exposed to seawater. The types of organisms and their probability and rate of occurrence are based on several environmental conditions:

- Water temperature
- Humidity

- Sunlight exposure (variations/cycles)
- Wood type
- Wood preservative (when used)

In some areas, the speed of deterioration is surprisingly fast, particularly in a warm air/water environment. A summary of the most common organisms and their associated effect on wood structures is provided in Section 3.7.2, Deterioration of Timber Structures.

The rubber component of the DM-29 wood seal and the newer all-rubber seal configuration (Figure 3-72) are not normally attacked by marine organisms, and are thus considered immune to this problem.

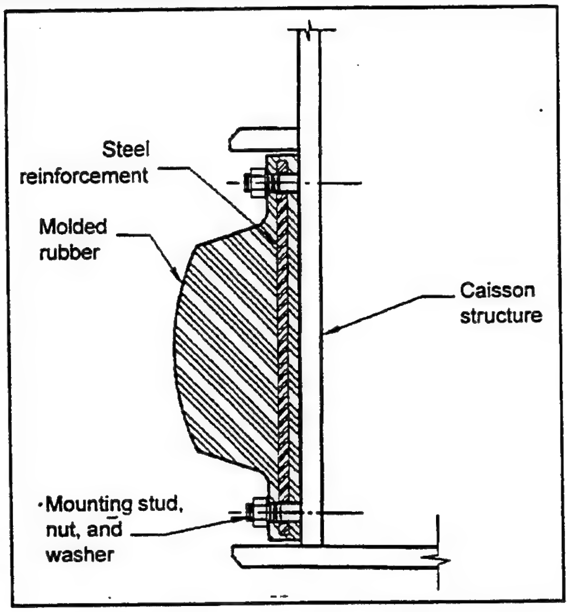



Figure 3-72. Steel-reinforced, all-rubber caisson seal assembly.

### 3.10.5 Drydock Seat Deterioration

As with the drydock structure and caisson assembly, deterioration of the drydock seat due to impact and exposure to seawater may occur. The condition of the seat and its resistance to deterioration will vary based on the amount and severity of impact events, seawater exposure, and composition of seat surface (steel, concrete, or stone).

In cases where an inner and outer seat exist and where the caisson has been positioned at the inner seat, the outer seat can become fouled with marine growth. This condition must be rectified before the caisson can be positioned at the outer seat. A similar situation occurs before superflood-ing. The inward facing seat is normally exposed to seawater and marine fouling and must be inspected and cleaned prior to superflooding the drydock to ensure an adequate seal.

### 3.10.6 Typical Inspection Procedure

Underwater inspection of the graving drydock structure, caisson assembly, and associated components will most likely be accomplished at incremental component levels and not in a fixed, sequential order, as inspection requirements and frequency vary with each drydock component. A typical underwater inspection of a drydock facility will involve inspection of the drydock structure, caisson assembly, caisson seal, drydock seat, stop logs, and drydock sill area.

Procedures for underwater inspection of steel, concrete, timber, and stone masonry waterfront structures are discussed in Sections 3.4.3, 3.6.3, 3.7.3, and 3.8.3,

respectively. In general, these inspection procedures are applicable to graving drydocks, caisson assemblies, and associated components.

### 3.10.7 Equipment and Tools Required

Equipment and tool requirements for inspection of waterfront steel, concrete, timber, and stone masonry structures are discussed in Sections 3.4.5, 3.6.4, 3.7.4, and 3.8.4, respectively. In general, these requirements are applicable to graving drydocks, caisson assemblies, and associated components.

Figure 5-20 illustrates a combination coating/cathodic protection system for a steel sheet pile wall.

Table 5-20 provides planning and estimating data for maintenance of steel sheet pile walls using cathodic protection.

## 5.7 CONCRETE PILES

As described in Section 3.6, there are many causes of deterioration of concrete piles in the marine environment. There are also many methods for the repair and maintenance of concrete piles. These repair and maintenance methods fall into four general categories:

- Concrete encasement
- Epoxy patching/injection
- Replacement
- Wrapping

Shotcreting may be used for restoring concrete piles above the waterline, but this method requires special equipment and training and is not normally performed by the UCTs.

### 5.7.1 Concrete Encasement

Concrete encasement is generally used for concrete piles that have not deteriorated to the point that the pile is not intact and the steel reinforcing is in good shape. In some cases where the pile is broken, the reinforcing steel may be parallel spliced by welding as directed by engineering.

The general procedures for repairing concrete using concrete encasement are described in Table 5-21. They are essentially the same as for steel piles as described in Section 5.5.1. Planning and estimating data are included in Table 5-22. Some general information is provided below. Refer to the figures in Table 5-21 for an outline of the procedure.

The repair procedure involves:

- Cleaning the pile
- Installing wire reinforcing, spacers, and forms about the pile
- Pouring concrete to fill the space between the pile and the form

Nonmetallic spacers are used to maintain a uniform 4-inch-thick concrete coating and ensure a minimum (2 inches) distance between pile and reinforcing steel and reinforcing and form. Form manufacturer's provide detailed instructions relating to installing their specific designs.

Both flexible and rigid forms are available and manufacturers are listed in Appendix A. Forms may also be fabricated from materials on site. Selecting the form type depends primarily on availability and choice of the designer or construction crew.

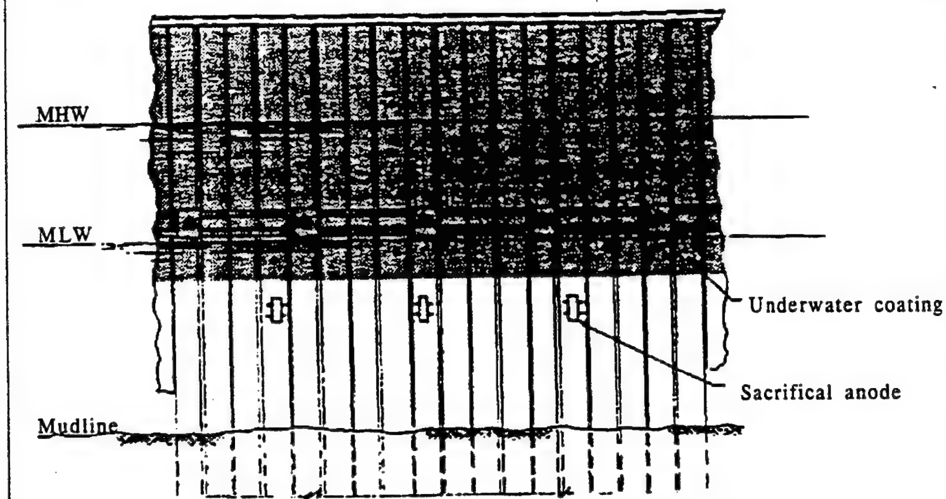



Figure 5-20. Coating/cathodic protection of steel sheet pile wall.

Table 5-20
Planning and Estimating Data for Steel Sheet Pile Maintenance
Using Cathodic Protection

Description of Task: Install a sacrificial anode on a steel sheet pile below low waterline. A 40-pound anode is attached to the pile with a welded connection.

Size of Crew: Two divers, one laborer.

Special Training Requirements: Familiarity with removal of marine growth, underwater lifting procedures, and light underwater welding.

Equipment Requirements: High-pressure waterblaster, hydraulic grinder with Barnacle Buster attachment, high-pressure pump for waterblaster, hydraulic power unit, welding machine with leads, welding torch, stud driver, lift bags/rigging gear, float stage or work platform.

Procedure: Clean area to bare metal where anode is to be attached. Anode may be attached by bolting, welding, or using a stud driver.

Productivity of Crew: 1 hour per anode.

Materials:

Anode - The number and spacing of the anodes must be determined by a cathodic protection engineer.

Table 5-21
Concrete Encasement of Concrete and Timber Piles

Problem: Precast concrete pile deteriorates due to sulfate attack, spalling, or cracks; or 10 to 50 percent of timber cross-sectional area has been lost due to marine borer attack.

Description of Repairs: Flexible or rigid forms together with reinforcing are used to produce a partial or complete concrete casing around damaged pile. Follow form manufacturer's directions. As required, excavate below mudline to expose deteriorated pile.

- Thoroughly clean pile of marine growth using scrapers, waterblaster, and sandblaster.

- Clean exposed rebar with wire brush. Remove deteriorated concrete, or wood, rust, and loose scale.

- Exposed/deteriorated reinforcing may be parallel spliced as directed by engineer.

- Install steel reinforcing around pile, use spacers to provide minimum 2-inch clearance between pile reinforcing and reinforcing form.

- Pump concrete using 15-psi minimum pressure through suitable pipes or hoses extending to lowest point. Keep discharge submerged in concrete.

Flexible Form - Closed Top:Figure 1 - Wrap form around pile, close zipper, secure form top and bottom to pile with mechanical fasteners or banding. Insert fill hose through fill port and project to bottom of bag.

Flexible Form Open Top Bag Figure 2 - Wrap the form around pile and suspend from deck structure or friction clamp using turnbuckles or chain. For timber piles, use spikes to hold up form. Secure form at bottom using mechanical fasteners or clamps. Insert fill hose to bottom and fill to overflowing. Cap top of form with epoxy or nonshrink grout to provide a 45-degree top.

For Rigid Form: Commercial forms (Figure 3) or forms fabricated using T&G (Figure 4), wood, fiberglass, plywood, or steel may be used. A base plate/clamp (Figure 3) must be fabricated to hold form in place and also provide a bottom seal for concrete during pour. Alternately, the soil at bottom of hole may be used as seal. Assemble form around pile and install bands at 12-inch intervals (or per instructions from manufacturer). The bands hold form in place during pour. Install band with fill fitting band atlowest location. Fill to overflow with concrete. Cap top of form with epoxy or nonshrink grout to provide a 45-degree top. Form and bands are removed after concrete is set (48 hours).

NOTE: Past construction operations used creosoted timber to provide chaffing protection from marine traffic or ice. Where environmental regulations prevent using creosote, substitute CCA or AZCA salt. When used, chaffing should extend from 1.5 feet above MLW to 2.5 feet below pile cap.Time between surface prep and concrete placement should be less than 36 hours in warm water and 72 hours in cool water.

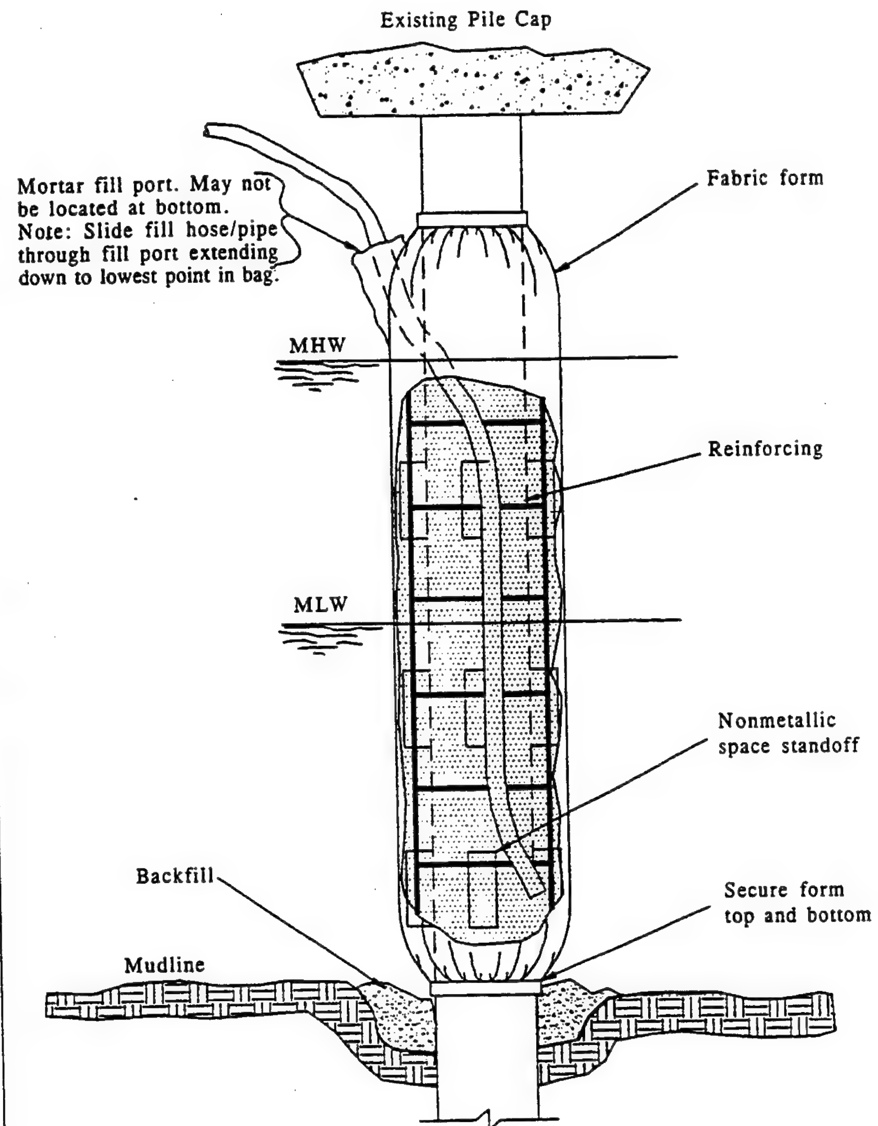



Table 5-21
Figure 1. Flexible form - Closed top.

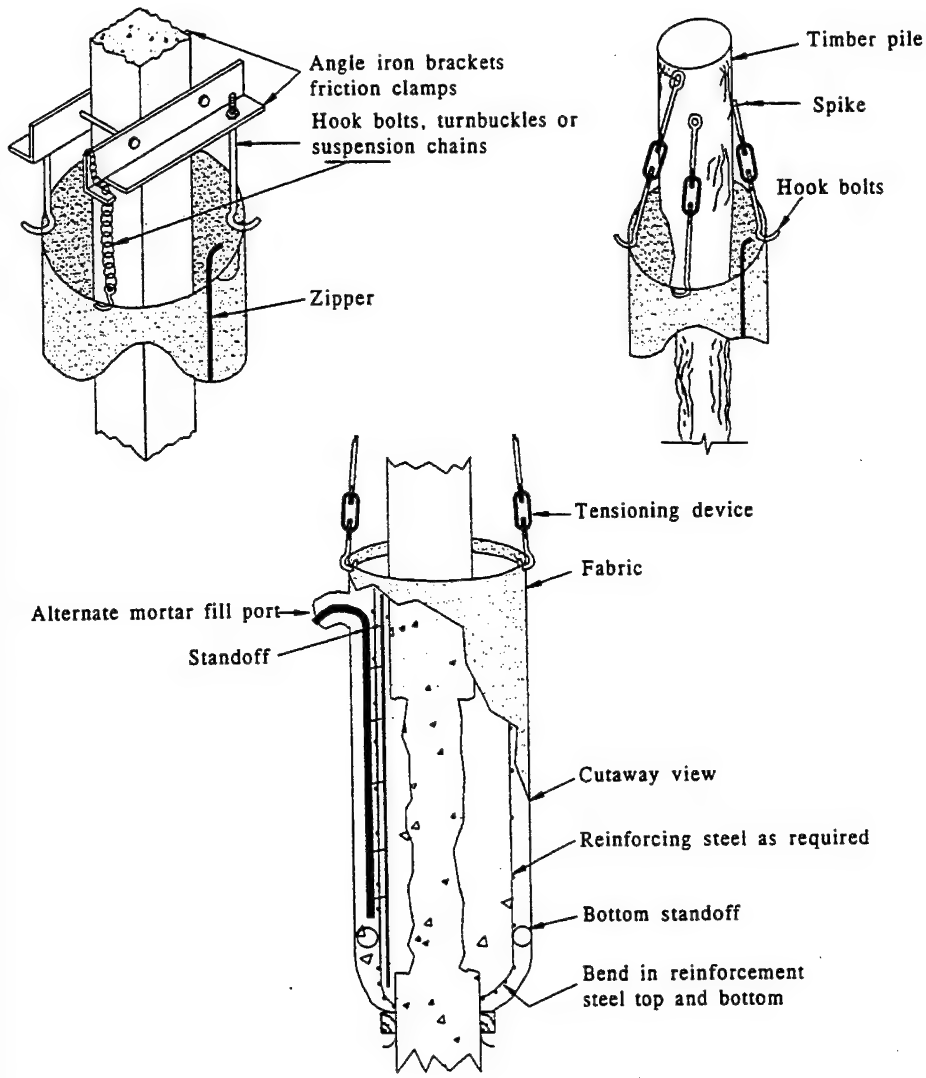



Table 5-21

Figure 2. Open top fabric form. Suspender detail and fabric hanging procedure.

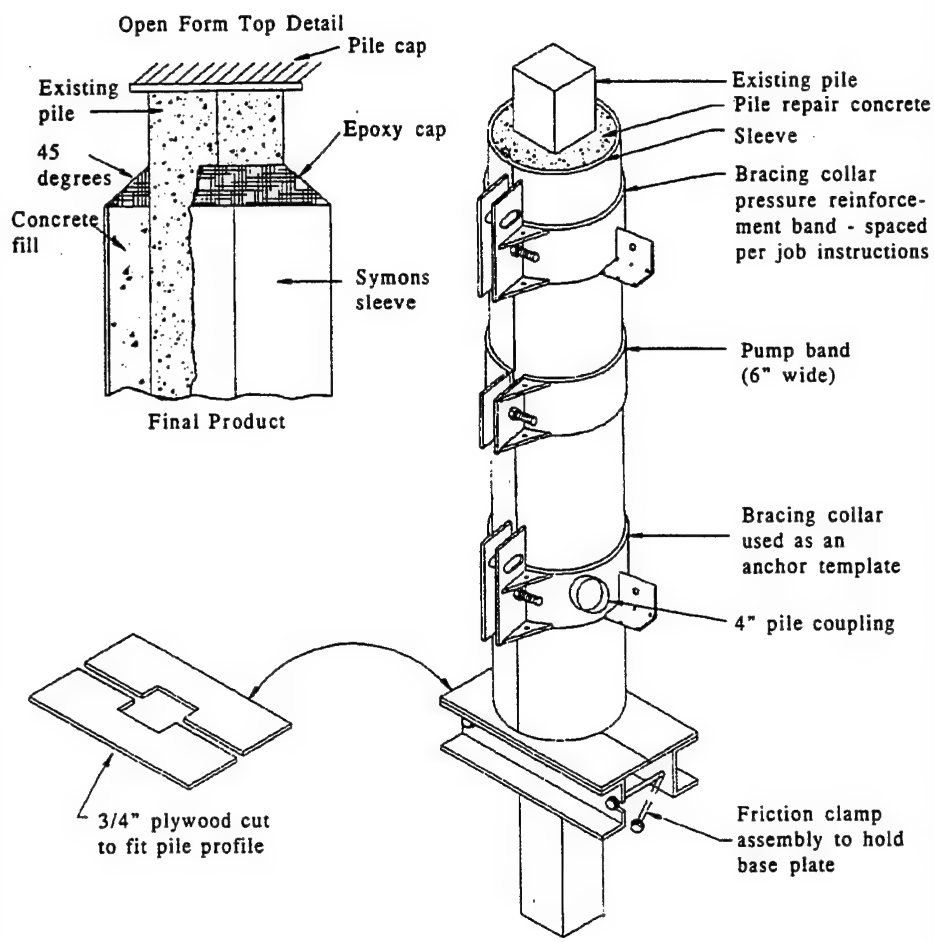



Table 5-21
Figure 3. Commercial rigid form.

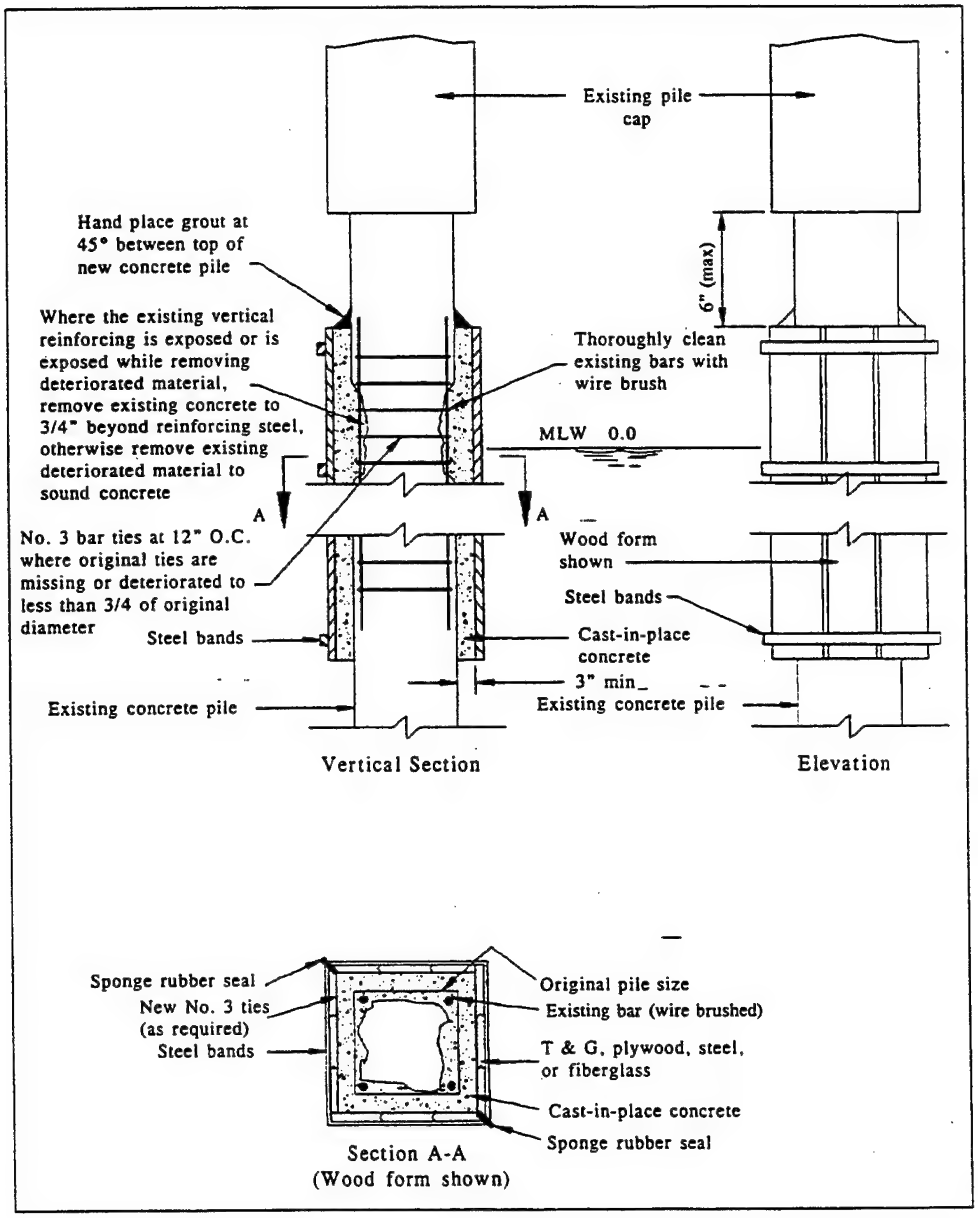



Table 5-21
Figure 4. Conventional T& G form detail (wood form shown).

Table 5-22
Planning and Estimating Data for Concrete Pile Repair
Using Concrete Encasement

Description of Task: Repair a deteriorated concrete pile by installing a concrete encasement from 1 foot above the high waterline to 1 foot below the mudline. The total length of encasement is 20 to 30 feet. Reinforcement of the pile is not required.

Size of Crew: Dive station, two laborers.

Training Requirements: Familiarity with the jacket to be used, concrete pump operation, jetting or air lifting procedures, and removal of marine growth.

Equipment Requirements: High-pressure waterblaster, hydraulic grinder with Barnacle Buster attachment, hydraulic power unit, concrete pump with adequate hose, concrete mixer (if ready-mix concrete is not available), jetting pump and hose, rigging equipment, float stage, scaffolding.

Productivity of Crew: 8 hours per pile repair.

Materials:

Form Material - When using proprietary forms, follow manufacturers' recommendations regarding lengths and diameter of forms, top and bottom closures, spacers, bands, straps, and special fittings. Provide 20 percent extra fittings for breakage if the forms are to be reused. Forms are ordered prefabricated in the required length and diameter. For flexible forms, allowance on the length must be made for extra fabric that may be required around blocking at the top and bottom of the jacket. Some proprietary systems require that different types of forms be used in the tidal and submerged zones.

Spacers - A conservative estimate of the number of spacers must be made. In calm water and with vertical piles, relatively few spacers will be required. Rough water and batter piles will require more spacers.

Wire Mesh Reinforcing - Usually 6x6-10/10 welded wire fabric is adequate. Calculate the width of wire fabric required based on its circumference, taking into consideration the thickness of the spacers between the pile and reinforcing and allowing a 9-inch overlap of the ends.

Concrete - To determine the amount of concrete required to fill the form, be conservative. When using flexible jackets allow for reduction of concrete volume due to loss of water through the permeable fabric, enlargement of the jacket caused by stretching, and waste. Usually an allowance of 10 percent extra concrete over the theoretically calculated quantity is sufficient.

• Flexible Forms - These forms are fabricated from a porous fabric and are usually left in place. Both open and closed top models are available. Open top fabric forms are suspended from the piling or overhead structure and are filled to overflowing. The concrete grout fill is topped with an epoxy cap or nonshrinking grout at a 45-degree angle as shown in Figure 5-17.

• Rigid Forms - These forms are open top and bottom. They are removed and reused after the concrete has completely cured. Rigid forms can be fabricated from material found at the construction site. A bottom seal and base plate are required to hold the form in place. A temporary friction clamp and base plate conforming to the pile contour is fabricated and attached to the pile at the bottom of the repair area (Figure 3 of Table 5-21). The rigid form rests on the base plate to form the bottom seal.

The piles and any exposed reinforcing steel must be cleaned of rust, scale, and marine life. Failure to do so would result in poor bond between the concrete and pile. It is also important to ensure that the concrete be poured within 72 hours following cleaning to ensure that new marine growth will not have formed on the pile surface. For warm waters, it may be necessary to further restrict this allowed time period.

Concrete should be pumped using 15-psi minimum pressure through a suitable pipe or hose which extends down to the lowest point in the form. It is important to start filling with the hose at the bottom and to keep the discharge submerged in concrete during the pouring operation. This procedure will prevent voids forming. Care must be taken when using the hose-tremie method as there is a tendency for the hose to become snagged on the wire mesh reinforcing. Some forms have built-in filling and overflow ports where the lower port is used for filling. Filling from a bottom port is preferred over extending the hose to the bottom of the form.

### 5.7.2 Epoxy Patching/Injection

Epoxy patching is used where localized spalling or cracking occurs. The deteriorated area must be thoroughly cleaned to allow the patch to adhere to the concrete. Once cleaned, the epoxy compound is mixed in accordance with the manu-facturer's instructions, forced into the spalled area, and worked smooth by hand. Epoxy will not stick to a wet surface so it must be kneaded to the surface to displace water. The patching epoxy used is described in Section 2.11.1. These epoxy patching compounds should be limited to thicknesses of less than 3/4 inch because of the difference in temperature-related expansion between the epoxy and underlying concrete. Thicker patches will deteriorate when subjected to temperature variations. Portland cement may be substituted for the epoxy grout, but it is more difficult to work with.

Cracks can be repaired by injecting low-viscosity epoxy mortar under pressure into the crack (Figure 5-21) as described in Section 2.11.3. This procedure is summarized in Table 5-23. The first step is to clean the area to be repaired. Plastic fill ports are available from manufacturers of epoxy injection systems. The plastic ports are attached to the crack entrance with an epoxy sealer. The crack surface is also sealed. The epoxy seal is allowed to cure overnight. The next day a low viscosity epoxy resin is injected through

Table 5-23
Concrete Crack Repair by Epoxy Grout Injection

Problem: Cracks caused by weathering, deterioration, or reinforcing steel corrosion allow water to penetrate the concrete

Description of Repairs: The technique involves forcing a suitable underwater grout to fill voids in cracks and displace existing water (Figure 1). Small cracks can be filled using hand-operated grout gun applicators while large cracks would require pressure grouting. Some epoxy grouts are hand molded into balls and then hand forced into the crack. Underwater grouts will not adhere directly to wet surfaces; the application method should strive to drive off the water at the grout-surface interface. Epoxy grouts adhere better to rough and clean surfaces.

- Thoroughly clean crack and adjacent surface by using waterblaster and mechanical tools - wire brush and scrapers, which will fit into the void. Remove loose material and marine growth. Clean any exposed reinforcing.

- Seal the full length of the surface of the crack using sealing tape or epoxy or similar grout pressed into the crack to form a dam. Use a fiberglass tape with a suitable underwater setting epoxy grout to seal the surface. Allow the seal to cure completely before proceeding.

- After the crack dam has cured, drill 1/2-inch or larger injection holes to the bottom of the crack and along the crack at 6-inch intervals. Size of hole depends injector tip.

- Insert nozzle of grout injector/dispenser into lowest drilled hole and inject flowable grout until it comes out the next higher hole. Proceed up the crack until all holes are filled.

Note: It may be necessary to add an extension tip to the nozzle to reach the end of the crack.

Materials: Manufacturers are listed in Appendix A.

- Select a flowable, low viscosity epoxy suitable for wet surfaces and underwater application for injection into hole. For temperatures below 55°F, use a polyester resin-based grout.

- For sealing grout, select a fast curing epoxy paste suitable for wet surface and underwater compatible that can be applied by hand or blade.

- Epoxy grout injection gun described in Section 2.11 can be used to dispense two-part epoxy grouts.

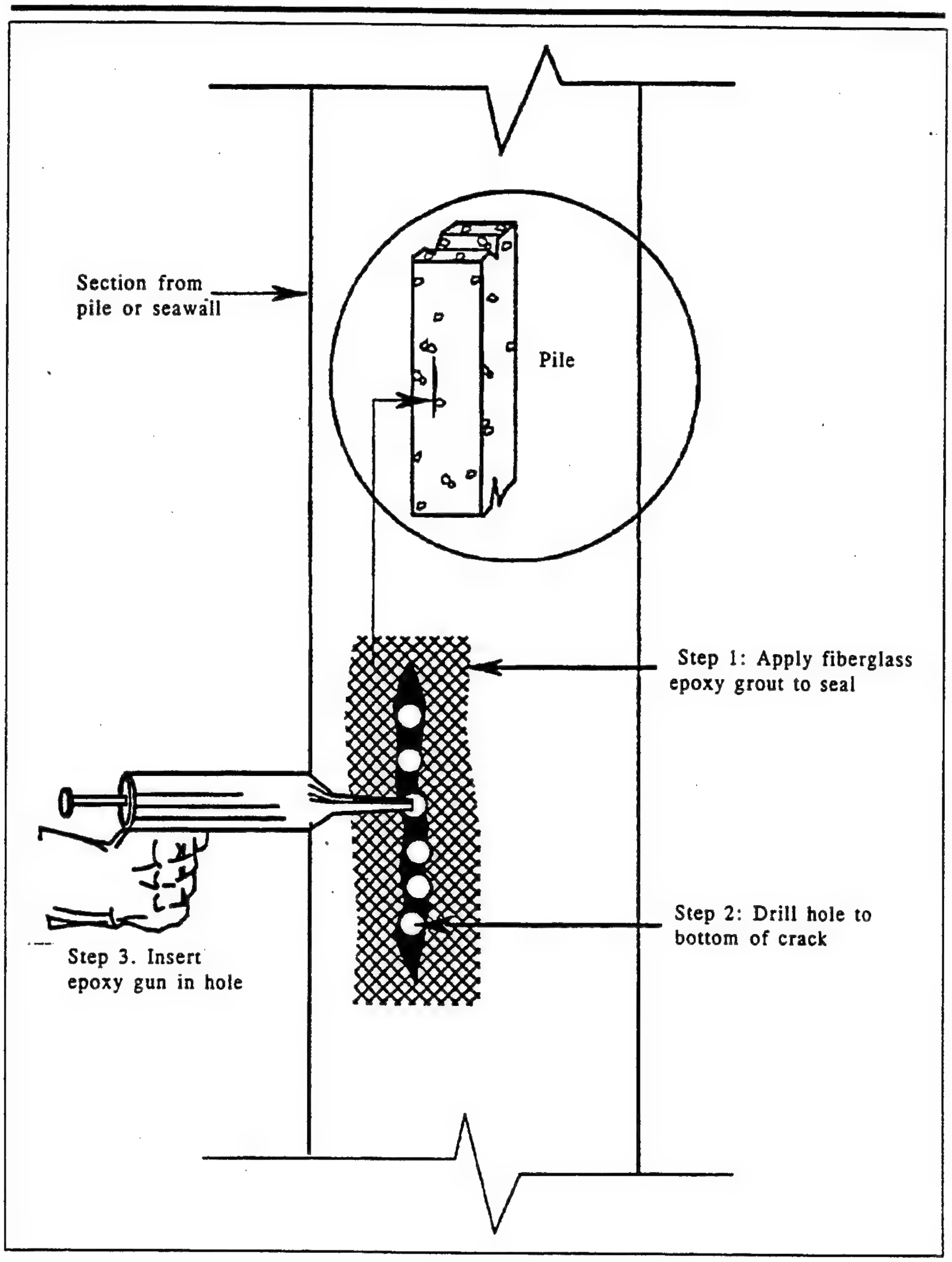



Table 5-23
Figure 1. How to repair concrete cracks with epoxy grout.

the ports to overflowing. After cure, the ports and excess surface materials are removed by grinding.

For deep cracks with wide entrances, fiberglass cloth in conjunction with an epoxy sealer are used to dam the surface. After the dam has cured, holes should be drilled into the crack every 6 inches along its length and used as injection ports so that the epoxy grout extends to the base of the crack. The epoxy grout is injected into the ports, starting at the bottom. After injection is complete, the port holes should be plugged to prevent the epoxy weeping out before setting.

Planning and estimating data for repair of concrete piles are provided in Table 5-24 (using epoxy patching) and in Table 5-25 (using epoxy injection).

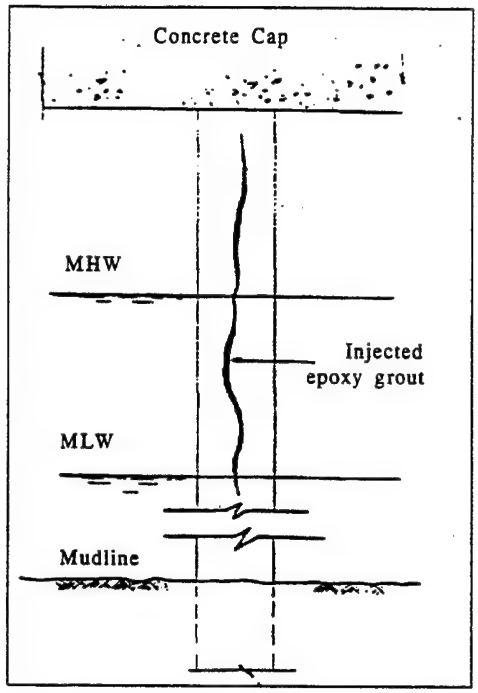



Figure 5-21
Epoxy grouting of concrete pile.

### 5.7.3 Replacement

A repair technique that may be economical in some situations consists of the replacement of the entire pile. Replacement pile techniques for concrete piles are the same as for steel piles and are described in Section 5.5.3.

Pile replacement techniques require heavy equipment that is not normally available to the UCTs and is usually carried out by others. NMCBs have heavy equipment to carry out pile replacement.

### 5.7.4 Wrapping

Piles wrapped with polyvinyl chloride (PVC) sheet are more resistant to abrasion and pitting by chemical action because there is no direct action by seawater on the pile. Pile wrapping techniques are described in Section 5.9.4.

Table 5-26 provides planning and estimating data for maintenance of concrete piles using wrapping.

## 5.8 CONCRETE SHEET PILE STRUCTURES

As described in Section 3.6.2, there are many causes of deterioration of concrete sheet piles and other precast or cast-in-place concrete structures in the marine environment. There are also many methods for the repair and maintenance of these structures. The repair methods fall into four general categories:

Table 5-24
Planning and Estimating Data for Concrete Pile Repair Using Epoxy Patching

Description of Task: Repair a deteriorated concrete pile by patching with hand-applied epoxy. Unit area to be repaired is 1 $ft^2$ underwater.

Size of Crew: Dive station, one laborer.

Special Training Requirements: Familiarity with procedures for removing marine growth and application of epoxy patching compounds underwater.

Equipment Requirements: High-pressure waterblaster, hydraulic grinder with Barnacle Buster attachment, high-pressure pump for waterblaster, hydraulic power unit, protective clothing for personnel handling the epoxy patching compound, float stage or work platform.

Productivity of Crew: 30 min/$ft^2$ underwater.

Materials:

Epoxy Patching Compound - Epoxy patching compounds are usually purchased in two-component kits, with an aggregate additive. A 1-gallon kit might include 1 gallon of each component plus aggregate, resulting in more than a 2-gallon yield. Patching coverage is measured in square feet per gallon. The required patching yield is obtained by taking the square footage to be covered and dividing by the square foot per gallon coverage rate.

Potential Problems: If water temperature is less than 60°F, proper adhesion to the pile may not occur. Skin irritation may occur if individual is sensitive to the epoxy material. Use protective gloves while mixing and applying epoxy materials.

Table 5-25
Planning and Estimating Data for Concrete Pile Repair Using Epoxy Injection

Description of Task: Repair a 6-inch-deep crack in a concrete pile by injecting low-viscosity neat epoxy grout into the crack. Total length of crack to be repaired is 10 feet.

Size of Crew: Dive station, one laborer.

Special Training Requirements: Familiarity with procedures for removing marine growth, the use of epoxy grout pump, and the use of injectable epoxy.

Equipment Requirements: High-pressure waterblaster, hydraulic grinder with Barnacle Buster attachment, hydraulic impact wrench with masonry bits, high-pressure pump for waterblaster, hydraulic power unit, protective clothing for personnel handling the epoxy compound, epoxy pump, float stage or work platform.

Productivity of Crew: 10 to 20 min/linear foot of crack.

Materials:

Low-Viscosity Epoxy Grout - Commercially available injectable epoxy grouts are usually purchased in two-component kits. Mixing proportions will vary, so manufacturers' instructions should be followed. The volume of the crack must be estimated by taking its length, average width, and average depth. An additional 25 percent should be added to allow for overfilling of the cracks and inaccuracies in estimating the size of the cracks.

Potential Problems: If water temperature is less than 60°F, proper adhesion to the pile may not occur. Skin irritation may occur if individual is sensitive to the epoxy material. Use protective gloves when mixing or applying epoxy materials.

Table 5-26
Planning and Estimating Data for Concrete Pile Maintenance Using Wrapping

Description of Task: Wrap a concrete pile with commercially available polyvinyl chloride (PVC) wrapping sheets from 1 foot above the high waterline to 2 feet below the low waterline. Total length to be wrapped is 10 to 20 feet.

Size of Crew: Dive station, one laborer.

Special Training Requirements: Familiarity with procedures for removal of marine growth and with installation procedure for the PVC pile wrap to be used.

Equipment Requirements: High-pressure waterblaster, hydraulic grinder with Barnacle Buster attachment, high-pressure pump for waterblaster, hydraulic power unit, special fastener tools for pile wrapping (dependent on manufacturer), float stage or work platform.

Productivity of Crew: 3 hours per pile.

Materials:

Polyvinyl Chloride Pile Wrap Unit - Commercially available PVC pile wraps are available in many prefabricated sizes to fit various pile sizes and lengths. Some have separate units for splash zone and underwater use, while others are a single unit. Manufacturers' literature should be consulted before ordering.

Strap and Special Fittings - Each pile wrap manufacturer has particular fasteners and special fittings. Manufacturers' literature should be consulted before ordering.

Potential Problems: Projections or sharp edges on the pile may puncture or tear the pile wrap, so care should be taken to remove such problems.

- Replacement/reconstruction
- Epoxy patching/injection
- Tieback system installation/replacement
- Wall load modification

Shotcreting may be used above the waterline, but this method requires special equipment and training and is not normally performed by UCTs.

5.8.1 Replacement/Reconstruction

Deterioration that affects the structural integrity of the concrete structure, such as missing lengths of reinforcing steel, should be repaired by replacement or reconstruction of sections of the structure. A method using pumped or tremied concrete is used when holes extend through the concrete section, or are larger than 1 $ft^2$ in area and over 4 inches deep, or are more than 1/2 $ft^2$ in area and deeper than the reinforcing steel. A description of the repair technique is provided in Table 5-27.

- All questionable quality concrete should be removed by hand or power tools.

- The edges of the hole should be square and the top rim should be sloped in the direction of the pour.

- Surfaces of the hole that are above water should be kept continually wet for several hours to allow partial saturation of the old concrete to take place to ensure proper curing of the new concrete.

- Just prior to placement of the new concrete, the holes should be thoroughly cleaned of marine growth.

- Form work should be mortar tight and attached to the existing structure with form ties. Use cotton caulking or similar materials to make tight seal.

- Forms attached to flat surfaces are best supplied with chimneys. Chimneys allow concrete to be poured into the form and also permit the concrete to be forced into the hole by pressure applied to the chimney opening.

Place concrete underwater in accordance with the procedures described in Section 2.10. Remove form work the day after casting unless form removal would damage the newly placed concrete. The chimney projections should normally be removed after the third day, working up from the bottom; working down from the top tends to break concrete out of the repair.

Section replacement using cast-in-place concrete in open-top forms is a simpler operation. No special features are required in the form work, but they should be mortar tight when vibrated and, if possible, should give the concrete a finish similar to the adjacent areas.

Table 5-28 provides planning and estimating data for repairing concrete sheet pile structures using concrete reconstruction.

Table 5-27
Typical Concrete Wall Repair

Problem: Severely deteriorated areas require replacement with cast-in-place concrete.

Description of Repairs Defective section of wall is removed, surfaces and reinforcing steel are prepared, formwork constructed, and wall is restored with cast-in-place concrete.

Materials: Sand-cement mortar and portland cement concrete as designed by engineer, anti-washout admixture may be added for underwater repairs. Equipment used includes sandblasting and air-water jet cleaning equipment, concrete saw, air chipping hammer, air-suction gun for mortar if available, power vibrator and tamper, and conventional concrete placement tools.

Preparation:

- Use air hammer to remove unsound concrete to 3/4-inch minimum depth.
- Cut the top edge of the hole to slope as shown in Figure 1. If necessary to fill the hole from both sides, the slope of the cut should be modified accordingly.
- Cut the bottom and sides of the hole approximately square with the face of the wall. Spalling and featheredges can be avoided by having chippers work from both faces. All interior corners should be rounded to a minimum radius of 1 inch.
- Ensure a minimum of 1 inch clearance around all exposed reinforcing. Remove unnecessary tie wires.
- Clean all surfaces to bond to new concrete with wet sandblasting and water jet. Clean exposed steel with abrasive blasting. Epoxy coat steel where possible. Parallel splice-weld new steel where 25 percent or more of original diameter is lost.

Repair Procedures

- Construct front forms for patches so the concrete can be placed in lifts not more than 12 inches high. The back form can be built in one piece. Fabricate and fit all forms before concrete placement is started.
- Chimneys (accesses) may be required at more than one level. In some cases, a chimney may be necessary on both sides of the wall or beam. In all cases, the chimney should extend the full width of the hole.
- Ensure forms are substantially constructed so that pressure can be applied.
- Ensure forms are mortar tight at all joints between adjacent sections, between the forms and concrete, and at the tie-bolt holes to prevent the loss of mortar. Twisted or stranded caulking cotton, folded canvas strips, or similar material should be placed between the joints as the forms are assembled.
- Surfaces to receive new concrete should be kept wet for several hours prior to placement.
- Immediately before placing the front section of form for each lift, coat the surface of the old concrete with a 1/8-inch-thick layer of mortar. This mortar should have the same sand and cement content and the same water-cement ratio as the new concrete. The surface should be damp, but not wet. The mortar can be applied with an air-suction gun, by brushing, or being rubbed into the surface by hand encased in a rubber glove.
- Place concrete immediately. Ensure thorough compaction by vibration/tamping.
- Forms may be removed the day after placement. If chimneys were used, remove the remaining projections the second day working up from the bottom.
- Thoroughly moist-cure the new concrete.
- Finish with a wood float finish. Tool or chamfer edges and corners as required.

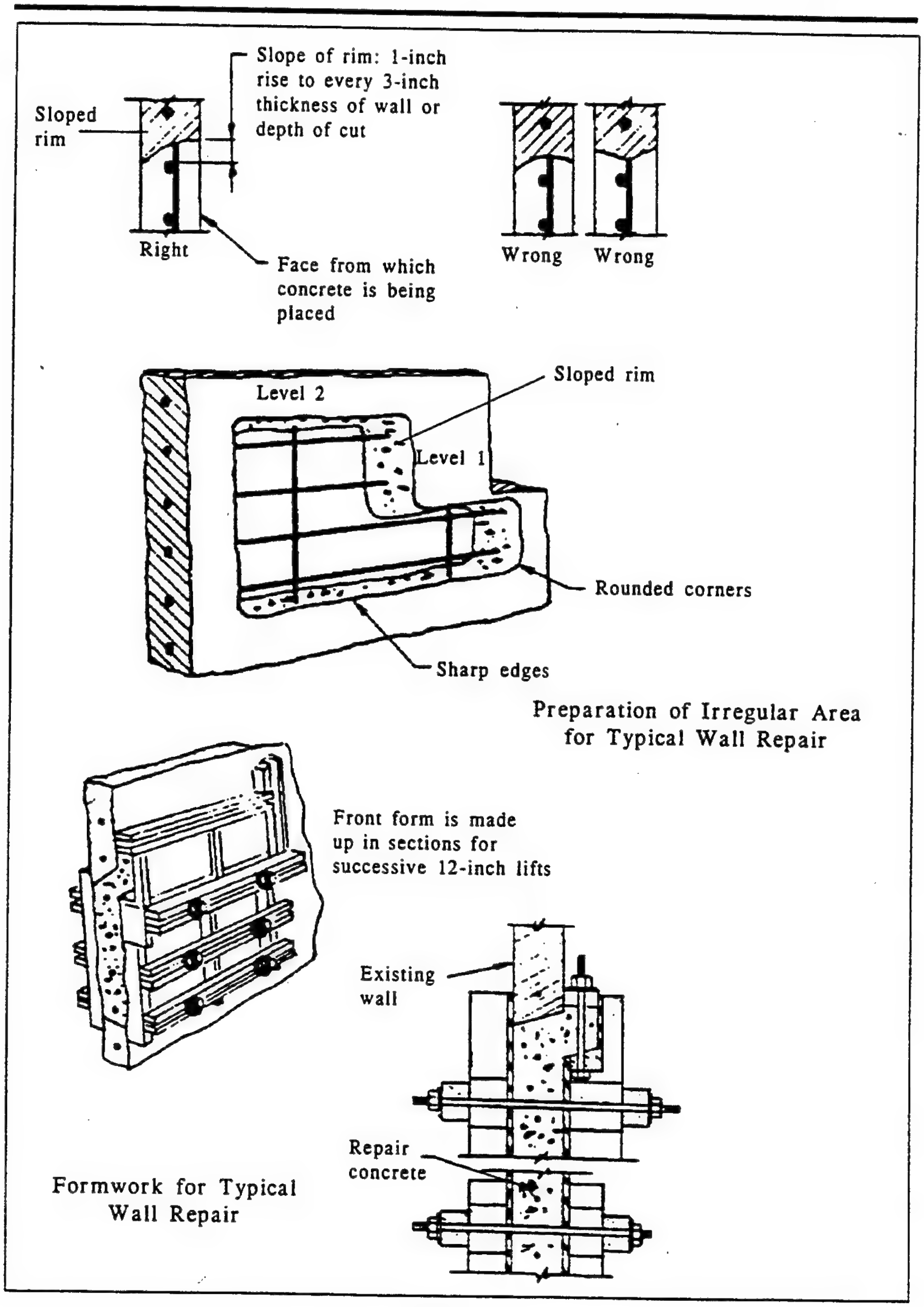



Table 5-27
Figure 1. Repair of concrete wall.

Table 5-28
Planning and Estimating Data for Concrete Sheet Pile Repair
Using Concrete Reconstruction

Description of Task: Repair a severely deteriorated 12-inch-thick concrete sheet pile wall by reconstructing the deteriorated portion with cast-in-place concrete. Unit area to be reconstructed is 10 $ft^2$. Access to the wall is from both sides.

Size of Crew: Dive station, two laborers.

Special Training Requirements: Familiarity with procedures for removal of marine growth, concrete pump/truck operations, concrete cutting/chipping machine operation, concrete formwork fabrication and installation.

Equipment Requirements: High-pressure waterblaster, hydraulic grinder with Barnacle Buster attachment, high-pressure pump for waterblaster, hydraulic power unit, hydraulic drill with bits, hydraulic chipping hammer, hydraulic hammer drill with bits, oxygen arc cutting equipment, rigging equipment, float stage or work platform.

Productivity of Crew: 40 hours per 10-$ft^2$ patch.

Materials:

Concrete Form Material - The concrete form is required for casting the repair section. The formwork must exist on both the front and back of the repair section and should extend a minimum of 3 inches over the existing sound concrete. The frontal plus the rear area of the repair plus the form overlap is the required area of formwork.

Steel Reinforcing Bars - Steel reinforcing bars must be placed in the repair section to replace those bars that were removed. Estimates can be made by measuring the gaps between sections of sound reinforcing steel. Allow for sufficient lap of new bars with existing bars.

Concrete - Concrete quantity is determined by the volume of the area to be filled. A conservative estimate should include an allowance of 10 percent extra concrete over the theoretically calculated quantity for waste and overfill.

Potential Problems: There may be difficulty in matching the existing concrete. Care must be taken in removing the forms; removing them too early will result in the concrete sagging at the top and bulging at the bottom. Forms must be dewatered before pouring concrete, if possible, or underwater concrete placement techniques must be used.

### 5.8.2 Tieback System Installation/Replacement

Movement of concrete walls can be arrested by installing a new tieback system or replacing the defective elements of an existing system. This repair/maintenance technique is explained in Section 5.6.4.

Tieback system installation/replacement requires major specialized equipment and is usually carried out by shore-based construction forces.

### 5.8.3 Wall Load Modification

Concrete wall movement can be arrested by changing the in-place loading on the wall, which is discussed in Section 5.6.5.

Wall load modification is usually carried out by shore-based construction forces.

## 5.9 TIMBER PILES

As described in Section 3.7, there are many causes of deterioration of timber piles in the marine environment. There are also many methods for the repair and maintenance of timber piles. Repair and maintenance methods fall into four categories:

- Concrete encasement
- Partial replacement
- Complete replacement
- Wrapping

### 5.9.1 Concrete Encasement

Concrete encasement can be used for timber piles that have not deteriorated to less than 50 percent of their cross-section area. The general procedures for repairing timber using concrete encasement are described in Table 5-21. They are essentially the same as for steel piles as described in Section 5.5.1. Planning and estimating data are included in Table 5-29. Some general information is provided below. Refer to the figures in Table 5-21 for an outline of the procedure. The repair procedure involves:

- Cleaning the pile
- Installing reinforcing, spacers, and forms about the pile
- Pouring concrete to fill the space between the pile and the form

Nonmetallic spacers are used to maintain a uniform concrete thickness and ensure a minimum (2 inches) distance between pile and reinforcing steel and reinforcing and form. Each form manufacturer provides detailed instructions relating to installing their specific designs.

Both flexible and rigid forms are available and manufacturers are listed in Appendix A. Forms may also be fabricated from materials on site. Selecting the form depends primarily on availability and choice of the designer or construction crew.

# Appendix B

# Mooring Design and Inspection Criteria

By

William Seelig

**NAVAL FACILITIES ENGINEERING SERVICE CENTER**
**Port Hueneme, California 93043-4370**

**Technical Report**
**TR-6009-OCN**

# MOORING DESIGN AND INSPECTION CRITERIA

by

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April 1999

Report Prepared for:
California State Lands Commission

## EXECUTIVE SUMMARY

The State of California is in the process of reviewing and formulating various design and inspection criteria for waterfront facilities. The Naval Facilities Engineering Command (NFESC) was invited to provide input, due to the U.S. Navy's experience and expertise.

In this report various commercial criteria are compared to MIL-HDBL-1026/4 "Mooring Design" (draft of 1998) and recommendations are made. This manual was designed for all classes of ships, including tankers. The State of California may want to consider adopting or incorporating this manual into their criteria.

Mooring analyses tools, a U.S. Navy ships' database, a climate database and a facilities database are being designed to work with MIL-HDBK-1026/4. This will allow the user to quickly and easily perform computations with a minimum of input. The State of California may wish to participate in development of these items.

CONTENTS

# MOORING DESIGN AND INSPECTION CRITERIA

By

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## 1.0 INTRODUCTION

It is vitally important that ships remain safely moored when in port. A single accident can result in tens or hundreds of millions of dollars in cost, disastrous environmental problems and a potentially huge loss of life. Proper mooring design, construction, inspection and operation can fortunately minimize the possibility of accidents. Fortunately, the cost of proper facilities is only a tiny fraction, for example, of the cost of a single ship and great progress has been made in recent years in improving safety. For example, computer methods and understanding of mooring technology have improved design methods. At the same time many years of practical experience and successful operation provide valuable insight.

In order to provide safe facilities, the California State Land Commission is in progress of reviewing facility design and inspection criteria for waterfront facilities. The goal of this review is to develop a comprehensive set of commercial standards.

The Naval Facilities Engineering Service Center (NFESC) was invited to participate in this development, because of NFESC's expertise and the Navy's extensive experience with a wide variety of waterfront facilities.

## 1.1 PURPOSE

The purpose of this report is to document and make recommendations on mooring design and inspection criteria. The Navy has recently completed a draft of "Mooring Design" MIL-HDBK-1026/4 (Seelig ed. of 1998) that addresses many of the items of interest. In this report the Navy standards are compared with various commercial codes. Examples are shown that compare the codes and recommendations are made.

## 2.0 CRITERIA

Criteria are provided for design and inspection of mooring facilities. The major emphasis of the criteria are for 'fixed' mooring facilities (i.e. ships at piers and wharves).

## 2.1 U.S. NAVY CRITERIA

The U.S. Navy owns ships and mooring facilities throughout the world, included facilities for tankers and similar ships. In the past, different criteria documents were provided for ship mooring systems and facilities mooring systems. However, in 1997-1998 all the criteria were updated and combined into MIL-HDBK-1026/4 "Mooring Design" (Seelig, ed. 1998). This handbook is intended for all classes of ships, including tankers. Appendix A includes Sections 3 and 4 of the handbook, which provides mooring design and inspection criteria, as well as methods for calculating wind and current forces/moments.

A key development provided in MIL-HDBK-1026/4 is the concept of *Mooring Service Type*. The U.S. Navy provides four types of mooring service, as shown in Table 6 (page 2-5) of Appendix A. These types of mooring are ranked from lowest to highest risk of a storm striking with a ship in the mooring. Design criteria are specified with each *Mooring Service Type* to minimize the risk of an accident.

*Mooring Service Types I*&II take care of cases with a ship moored one month or less, which is primarily the case at fuel facilities. Design criteria for these types of service are given in Table 7 (page 2-7) of Appendix A, which are shown in Table 2.1.

The wind criteria for design of this service type range from a 30-second wind speed of 33 knots to a wind with a return interval of R=25 years, up to 75 mph. MIL-HDBK-1026/4 uses ASCE 7-97 to specify design wind speeds. However, ASCE 7-95 also allows actual wind statistics to be used for site design, if adequate measured wind data is available for a site.

Water level, current and wave design criteria are shown in Table 2.1.

Locations of U.S. Navy design criteria from Section 3 of MIL-HDBK-1026/4 are given in Appendix A and locations of key information are given in Table 2.2.

If ships of similar size are moored alongside one another or nearby, then methods in Appendix A of MIL-HDBK-1026/4 can be used to determine environmental forces and moments on the ships.

### Table 2.1 FACILITY DESIGN CRITERIA FOR *MOORING SERVICE TYPES I&II*

| MOORING SERVICE TYPE | WIND* | CURRENT** | WATER LEVEL | WAVES |
|---|---|---|---|---|
| TYPE I | Less than 34 knots | 2 knots or less | mean lower low to mean higher high | P=1 or R=1 yr |
| TYPE II | P=0.04 (min.) R=25 yr (min.) $V_w$=64 knots (max.) | P=0.04 R=25 yr | extreme lower low to mean higher high | P=1 or R=1 yr |

*Use exposure D (American Society of Civil Engineers (ASCE) 7-95, Minimum Design Loads for Buildings and Other Structures; flat, unobstructed area exposed to wind flowing over open water for a distance of at least 1 mile or 1.61 km) for determining design wind speeds. Note that min. = minimum return interval or probability of exceedence used for design; max. = maximum wind speed used for design.

**To define the design water depth, use T/d=0.9 for flat keeled ships; for ships with non-flat hulls, that have sonar domes or other projections, take the ship draft, T, as the mean depth of the keel and determine the water depth, d, by adding 0.61 meter (2 feet) to the maximum navigation draft of the ship.

Table 2.2 KEY *MOORING SERVICE TYPE I* CRITERIA

| *CRITERIA* | *SOURCE** | *PAGE** |
|---|---|---|
| | ***Section 3*** | |
| Definitions of *Mooring Service Types* | Table 6 | 2-5 |
| Design criteria | Table 7 | 2-7 |
| Minimum quasi-static factors of safety | Table 9 | 2-10 |
| Ship motion criteria | Table 10 | 2-11 to14 |
| Quasi-static approach | Table 11 | 2-15 |
| Conditions requiring special analyses | Table 12 | 2-18 |
| Design considerations - facilities | Table 14 | 2-25 |
| Mooring operational design considerations | Table 18 | 2-42 |
| Inspections guidelines | Table 19 | 2-43 to 44 |
| Design recommendations | Table 20 | 2-46 to 47 |
| Quasi-static forces and moments on ships | ***Section 4*** | 2-48 |

*See Appendix A

## 2.2 OCIMF CRITERIA

Oil Companies International Marine Forum (OCIMF) has developed various criteria specifically intended for tankers. These include:

Oil Companies International Marine Forum (OCIMF), Mooring Equipment Guidelines, 1nd Edition, 1992.

Oil Companies International Marine Forum (OCIMF), Recommendations for Equipment Employed in the Mooring of Ships at Single Point Moorings, 3rd Edition, 1993.

Oil Companies International Marine Forum (OCIMF), Prediction of Wind and Current Loads on VLCCs, 2nd Edition, 1994.

Oil Companies International Marine Forum (OCIMF), Single Point Mooring Maintenance and Operations Guide, 2nd Edition, 1995.

Note that both the Navy and OCIMF have both recently changed their sign convention and reference coordinate systems to conform to the standard right-hand-rule and both use the same system. Both the Navy and OCIMF use the wind speed at 10 m as a reference. The Navy specifies a wind gust with a duration of 30-seconds, while OCIMF does not address wind gusts, but states "While vessels may respond to wind gusts of limited duration, the analysis of this subject is beyond the scope of this report."

## 2.3 OTHER CRITERIA

Various other sources address specific criteria. Some of these references include:

American Petroleum Institute, "Recommended Practice for Planning, Designing, and Constructing Tension Leg Platforms", API RP 2T, April 1, 1987.

American Petroleum Institute, "Analysis of Spread Mooring Systems for Floating Drilling Units", ANSI/API RP 2P-87, Approved July 12, 1993.

American Petroleum Institute, "Recommended Practice for Design, Analysis, and Maintenance of Moorings for Floating Production Systems", ANSI/API RP 2FP1-93, Approved April 13, 1994.

American Petroleum Institute, "Recommended Practice for Design and Analysis of Stationkeeping Systems for Floating Structures", API RP 2SK, 2nd Ed., Mar. 1, 1997.

Permanent International Association of Navigation Congresses, "Report of the International Commission for Improving the Design of Fender Systems", Supplement to Bulletin No. 45, 1984.

Permanent International Association of Navigation Congresses, "Criteria for Movements of Moored Ships in Harbours; A Practical Guide", Report of Working Group No. 24 of the Permanent Technical Committee II, Supplement to Bulletin No. 88, 1995.

These and similar references address various aspects of mooring. Some of the references are oriented towards offshore facilities, while others address specific aspects of a facility. In MIL-HDBK-1026/4, many references were reviewed and key items of interest were then considered and incorporated into the handbook.

## 3.0 COMPARISONS OF CRITERIA

### 3.1 GENERAL

MIL-HDBK-1026/4 (draft of 1998) was organized to be a comprehensive manual that addresses mooring design and inspection. Extensive U.S. Navy experience, together with a number of other references, were considered in preparing the manual. It was found that many of the other references did not specifically address waterfront 'fixed' mooring facilities (i.e. piers and wharfs) as extensively as the Navy methods. Therefore, portions of these other references were considered and then incorporated into the Navy manual, if appropriate.

The approach in MIL-HDBK-1026/4 was to use quasi-static methods and indicate conditions that may require further dynamic analysis. The handbook was designed to include almost any class of vessels, including tankers. A discussion of specific items is provided below.

<u>Risk</u>

A wind return interval of R=25 years was selected for *Mooring Service Type II* as providing reasonable risk. Facilities offering this type of service are often occupied. However, these vessels should be ready to go and leave the facility if extreme weather is predicted.

<u>Factors of Safety</u>

Factors of safety were selected so that mooring lines are the weak link, because lines are most easily tested and replaced when necessary. Facilities have slightly higher factors of safety, because they are designed to last longer and are more difficult to inspect and replace. Also, a facility may have a visit by some ship larger than originally envisioned when the facility was designed.

The design approach selects an extreme event. Calculations are performed assuming quasi-static conditions. Factors of safety are then selected to provide low risk at reasonable cost. They help account for typical factors, such as:

- mild dynamics of the system
- material wear
- variability in use
- uncertainty in calculations
- unknown factors

## 3.2 COMPARISONS OF FORCES

MIL-HDBK-1026/4 and OCIMF (1994) provide methods for estimating forces and moments on ships. Some of the key items concerning these methods are:

<u>MIL-HDBK-1026/4 method</u>:

For any vessel.

Uses 30-second duration wind speed.

Broadside wind drag coefficient considers elevation of hull and superstructure to come up with an effective drag coefficient.

Broadside current drag coefficient is a function of the hull shape and ratio of draft to water depth.

Longitudinal current drag is computed for the form, friction and propeller.

General shape functions are provided for wind and current forces/moments.

<u>OCIMF method</u>:

For tankers only.

Wind gust duration not specified.

Separate broadside wind coefficients given for loaded and light vessels.

Longitudinal current coefficient given.

Shape functions are given graphically for selected parameters. These are sometimes rather complex.

Selected comparison are shown to compare MIL-HDBK-1026/4 and OCIMF methods. Tankers are of special interest to the California State Lands Commission, so a 200,000 DWT tanker with principle dimensions given in Table 3.1 is used to illustrate the computed forces.

**Table 3.1 TYPICAL 200,000 DWT TANKER PARAMETES (after Wichers)**

| *PARAMETER* | *LOADED* | *LIGHT (BALLASTED)* |
|---|---|---|
| Length between perp. | 310 m | 310 m |
| Draft | 18.9 m | 7.56 m |
| Width | 47.17 m | 47.17 m |
| Disp. Volume | 234,994 $m^2$ | 88,956 $m^2$ |
| End-on Wind Area | 1362.4 $m^2$ | 1897.3 $m^2$ |
| Side Wind Area Hull | 3461.4 $m^2$ | 7095.9 $m^2$ |
| Side Wind Area Super. | 922 $m^2$ | 922 $m^2$ |
| Height of Hull | 10.8 m | 22.14 m |
| Height of Superstructure | 32.2 m | 43.64 |

Various force coefficients and forces are compared here to illustrate MIL-HDBK-1026/4 and OCIMF methods. In this report a drag coefficient is defined as a force divided by (0.5*density*exposed area*velocity squared).

Figure 3.1 and 3.2 show that longitudinal wind drag coefficients for 0-degrees (OCIMF Figure 2) and broadside wind drag coefficients for 90-degrees (OCIMF Figure 3) are similar to those computed using MIL-HDBK-1026/4.

A direct comparison of longitudinal forces for a 3-knot current shows that OCIMF and MIL-HDBK-1026 give similar results for a loaded tanker (Figure 3.3). The MIL-HDBK-1026/4 method predicts that a significant portion of the drag is due to the skin friction and propeller drag, so that a lightly loaded tanker has somewhat less current drag forces. OCIMF gives an unexpectedly smaller value for a lightly loaded tanker.

A comparison of broadside current drag coefficients shows the MIL-HDBK-1026/4 prediction fit the OCIMF (Figure 10) data very well, as shown in Figure 3.4.

The MIL-HDBK-1026/4 recommended shapes of forces and moments as a function of direction that are shown for wind in Figure 3.5 and for current in Figure 3.6. The OCIMF shape factors are much more complex and vary as a function of a number of parameters.

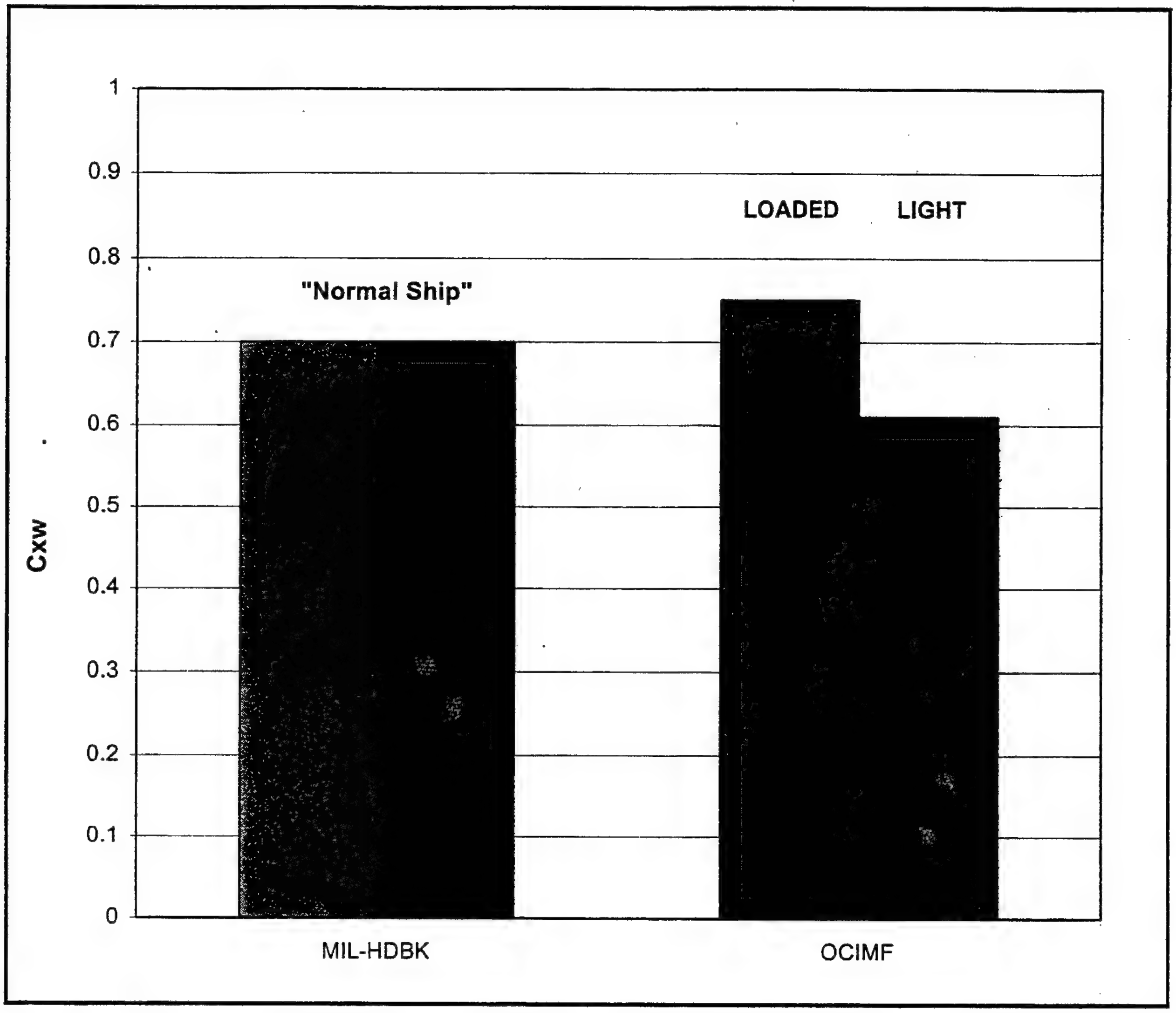


**Figure 3.1 WIND DRAG COEFFICIENTS FOR 0-DEGREES**

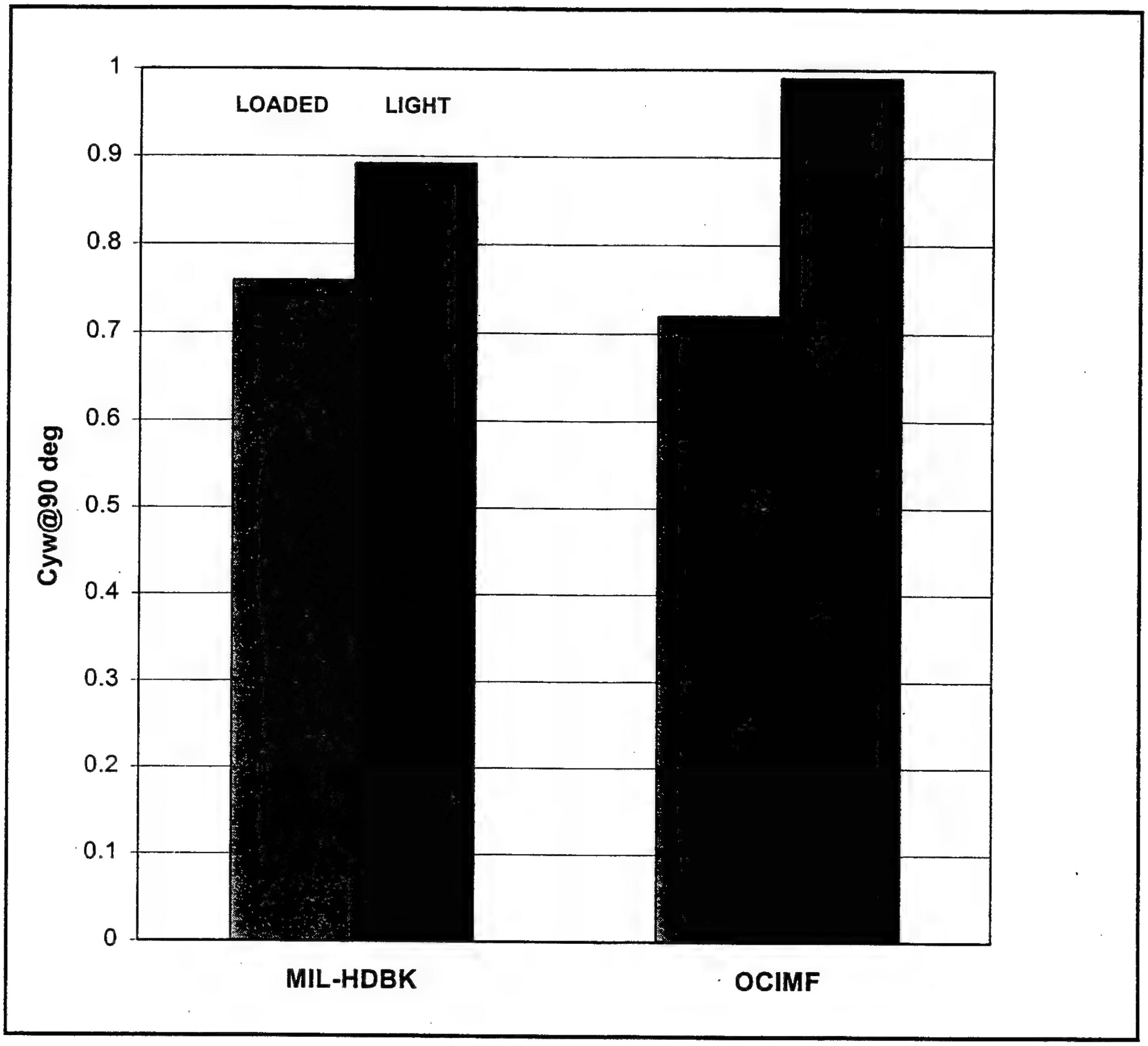


**Figure 3.2 BROADSIDE WIND DRAG COEFFICIENTS**

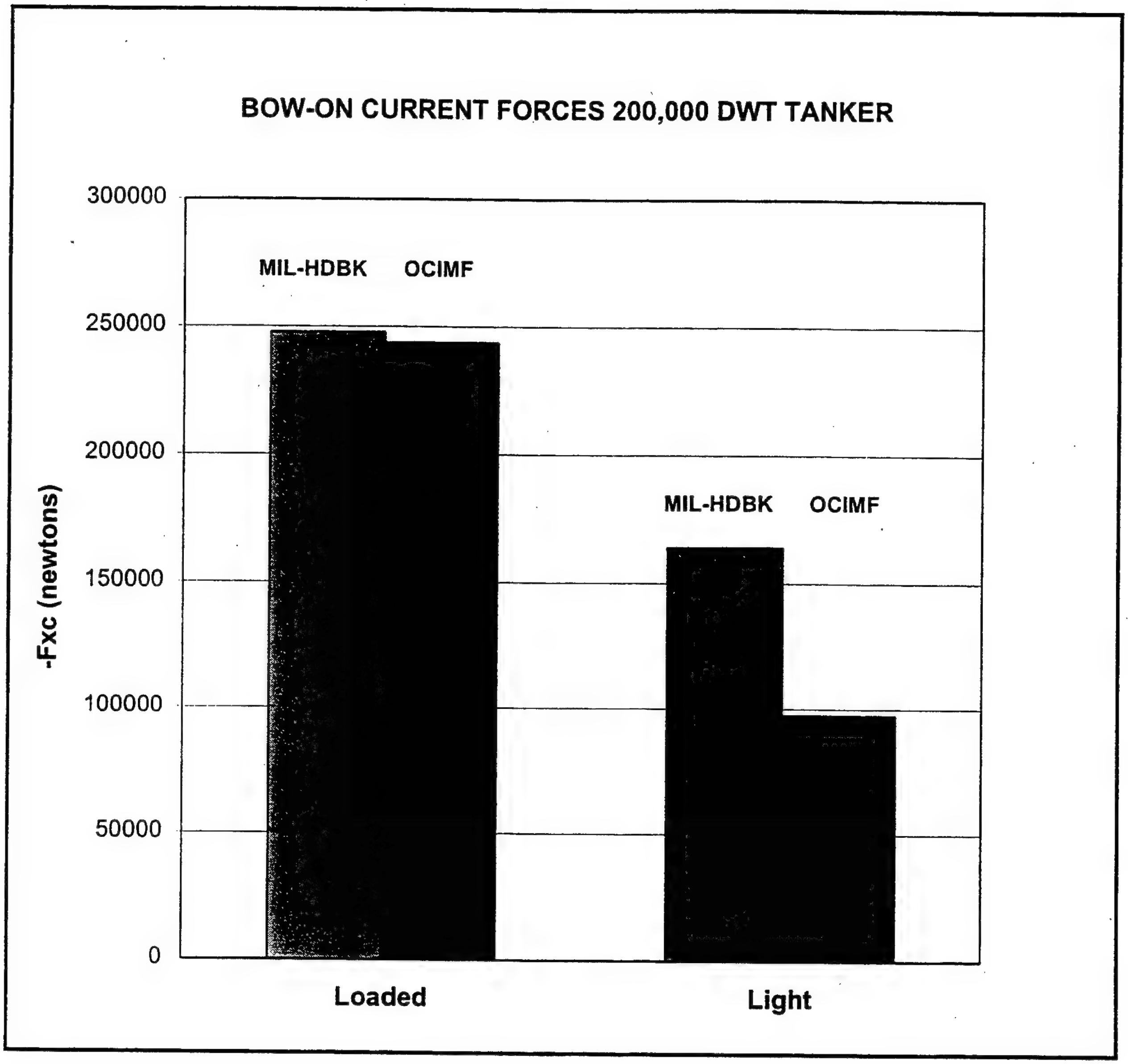


**Figure 3.3 END-ON CURRENT FORCES FOR A 3-KNOT CURRENT**

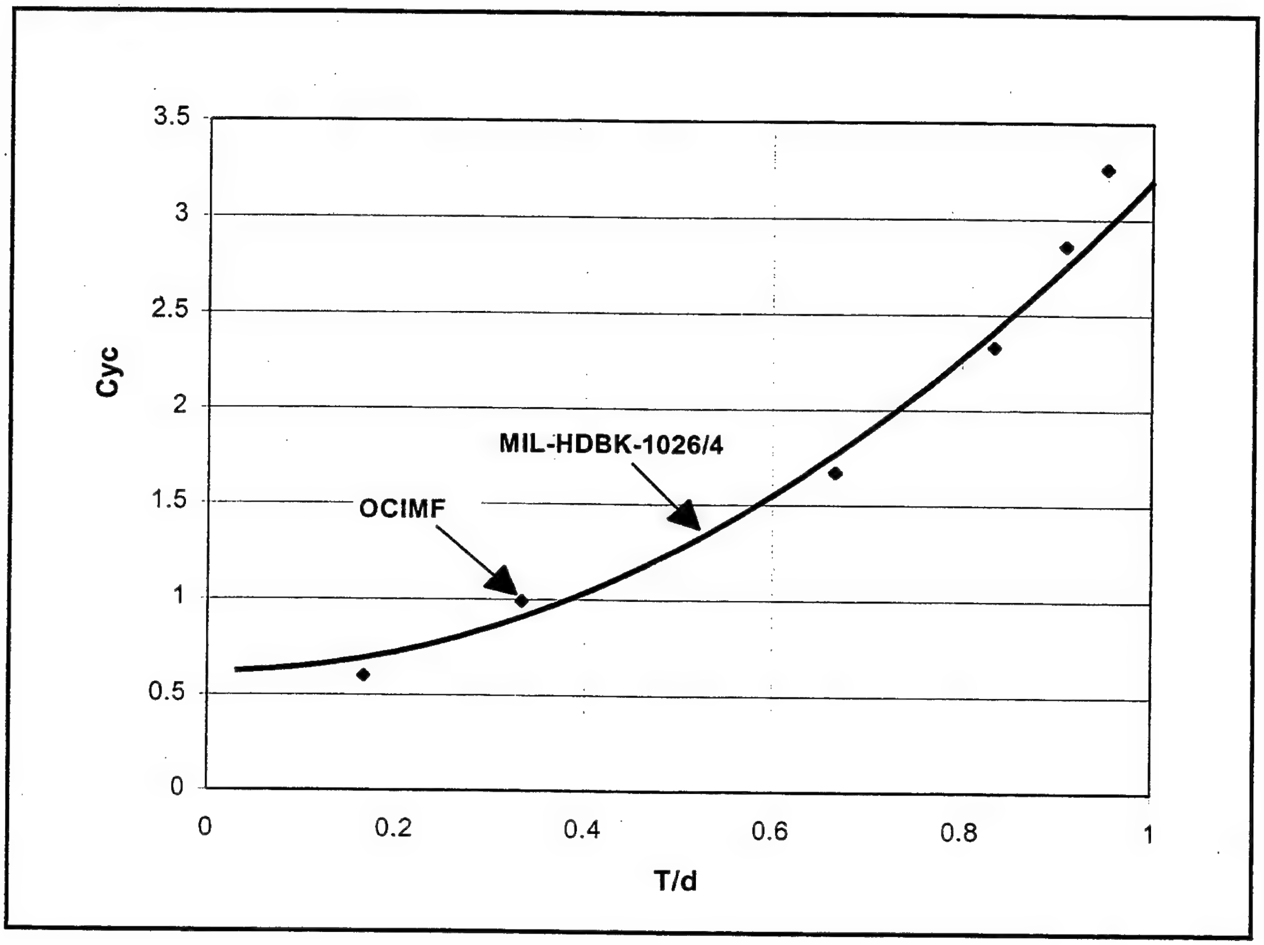


**Figure 3.4 BROADSIDE CURRENT DRAG COEFFICIENT PREDICTED FOR A 200,000 DWT TANKER**

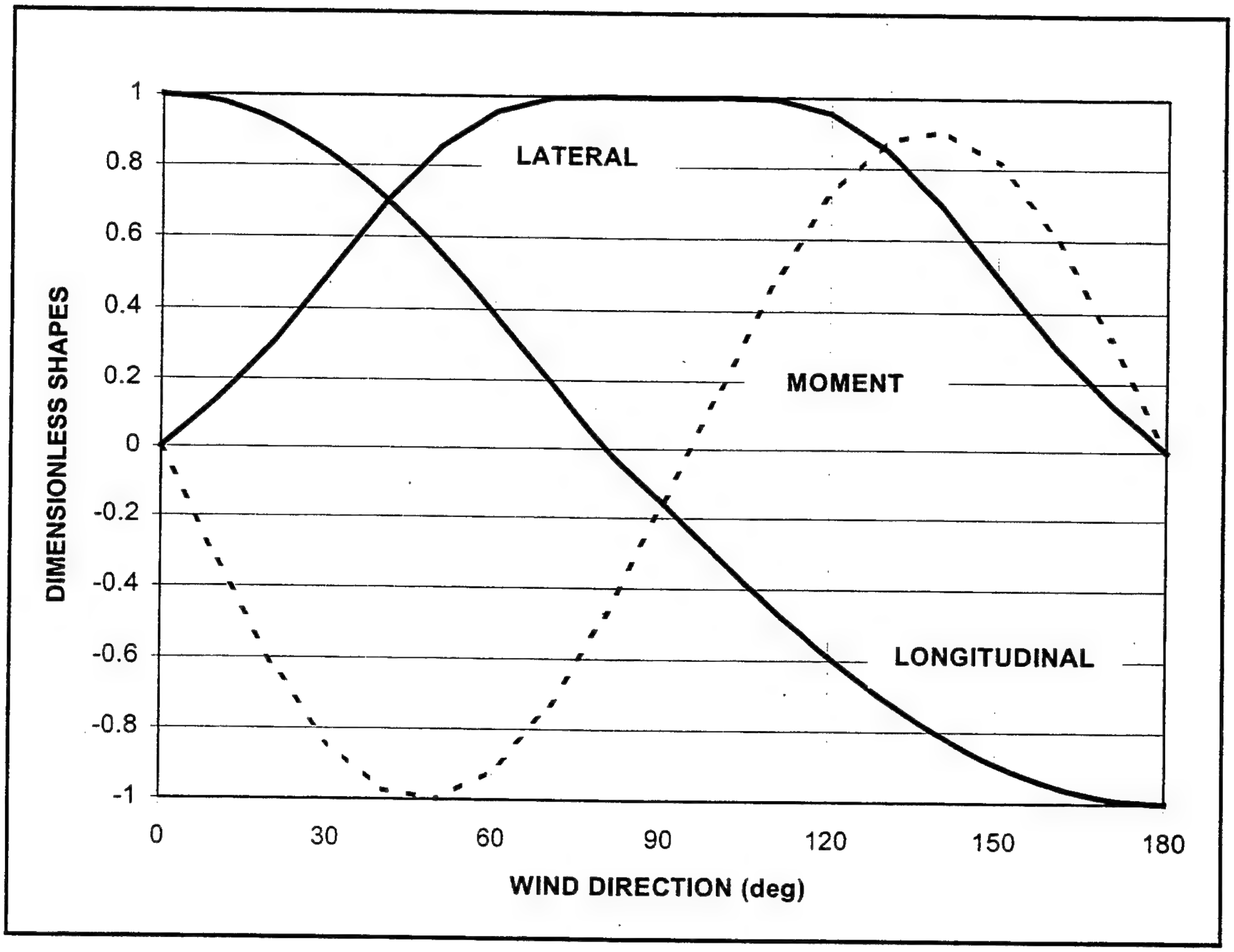


**Figure 3.5 WIND FORCE/MOMENT SHAPES**

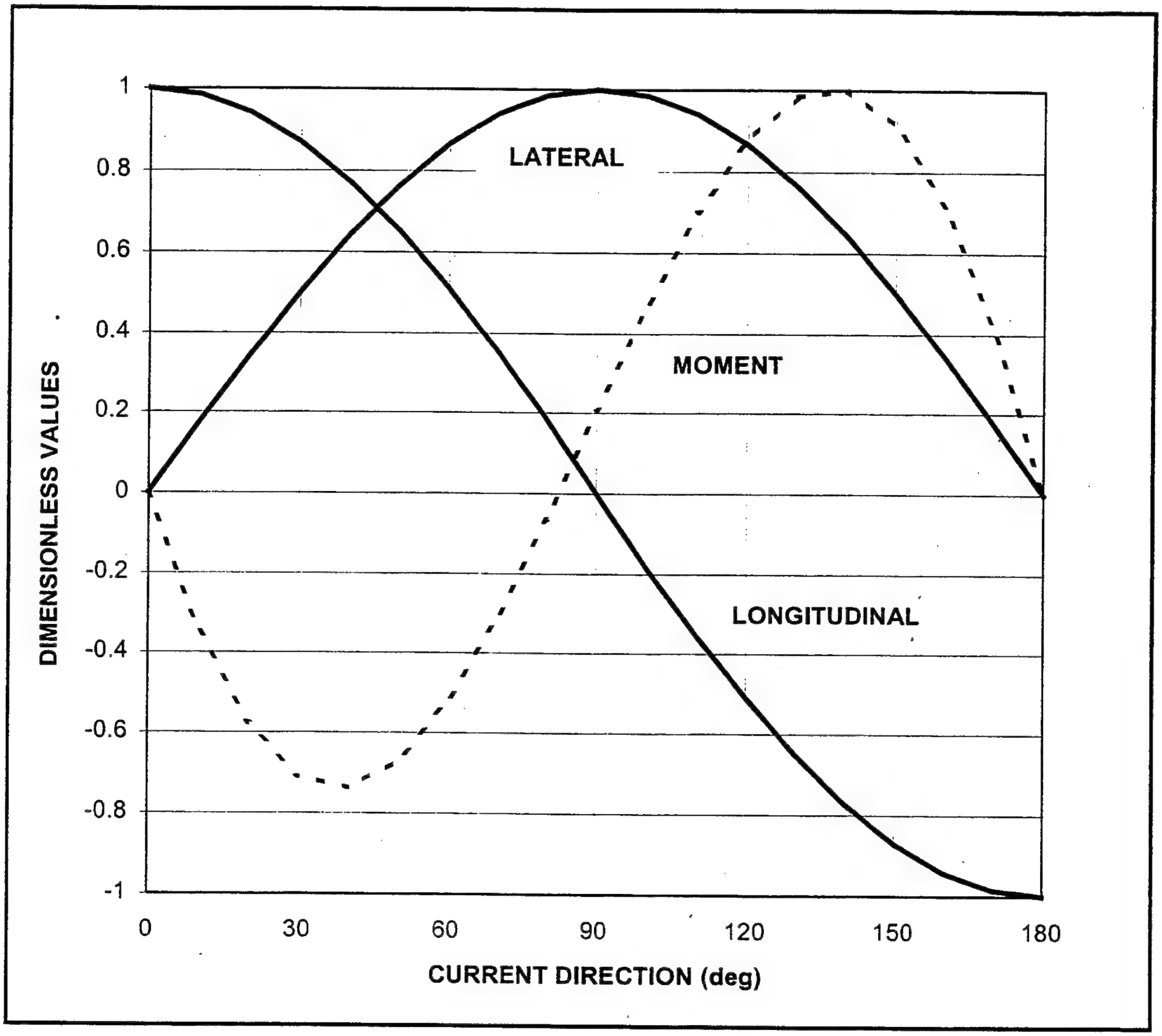


**Figure 3.6 CURRENTFORCE/MOMENT SHAPES**

## 4.0 DESIGN WIND SPEEDS

Environmental design criteria includes winds, tides, current and waves (if necessary). Water depths must also be known. Tides and currents can often be determined from NOAA records and the U.S. Army Corps of Engineers commonly has dredging records. Winds are then of special interest. *Mooring Service Type I* specifies a 30-second duration wind speed with a return interval of R=25 years (probability of P=0.04) with a minimum wind speed of 33 knots.

ASCE 7-95 gives a 3-second R=50 year design wind speed of 85 mph for all of California. This can be converted to a 30-second R=25 year design wind speed with Exposure D (wind flowing over open water for a distance of at least 1 mile or 1.61 km) to:

$$85 \text{ mph} * 0.87 * 1.086 * 0.93 = 74.68 \text{ mph}$$

More localized values of R=25 year 30-second duration wind speed values can be determined from taking R=50 fastest mile wind speeds from NUREG/CR-4801 and converting them using methods in ASCE 7-95 for R=25 years, 30-second duration and Exposure D. Table 4-1 gives these design wind speeds. Figures 4-1 and 4-2 show these design wind speeds in graphical form.

## Table 4-1. R=25 YEAR 30-SECOND EXPOSURE D WIND SPEEDS

| Location | (mph) |
|---|---|
| Alameda | 61.6 |
| Bakersfield | 59.6 |
| Bishop | 70.4 |
| Blue Canyon | 91.3 |
| Chula Vista/Brown | 42.0 |
| Coronodo/North Island | 58.6 |
| Edwards | 64.5 |
| El Centro | 75.2 |
| El Toro | 75.2 |
| Fairfield/Travis | 65.5 |
| Fresno | 50.5 |
| Imperial Beach/Ream | 58.6 |
| Inyokern/China Lake | 67.5 |
| Lemoore | 53.6 |
| Long Beach | 65.5 |
| Los Alamitos | 51.6 |
| Los Angeles Airport | 53.6 |
| Los Angeles City | 43.1 |
| Marysville/Bewale | 64.5 |
| Merced/Castle | 54.6 |
| Monterey | 64.5 |
| Mt. Tamalpias | 138.8 |
| Mt. Tamalpias | 135.1 |
| Oakland | 62.6 |
| Oxnard | 55.6 |
| Point Mugu | 67.5 |
| Point Reyes | 112.8 |
| Riverside/March | 51.6 |
| Sacramento | 69.4 |
| Scramento/Mather | 64.5 |
| Scramento/McClellan | 72.3 |
| San Bernadrino/Norton | 68.4 |
| San Clemente Island | 54.6 |
| San Diego | 64.5 |
| San Diego/Miramar | 51.6 |
| San Francisco City | 54.6 |
| San Francisco Airport | 72.3 |
| San Jose | 52.6 |
| San Nicholas Island | 56.6 |
| San Rafael/Hamilton | 68.4 |
| Sandberg | 98.8 |
| Santa Ana | 65.5 |
| Stockton | 68.4 |
| Sunnyvale/Moffett | 53.6 |
| Vandenberg | 55.6 |
| Victorville/George | 68.4 |
| Yuma, Arizona | 63.6 |

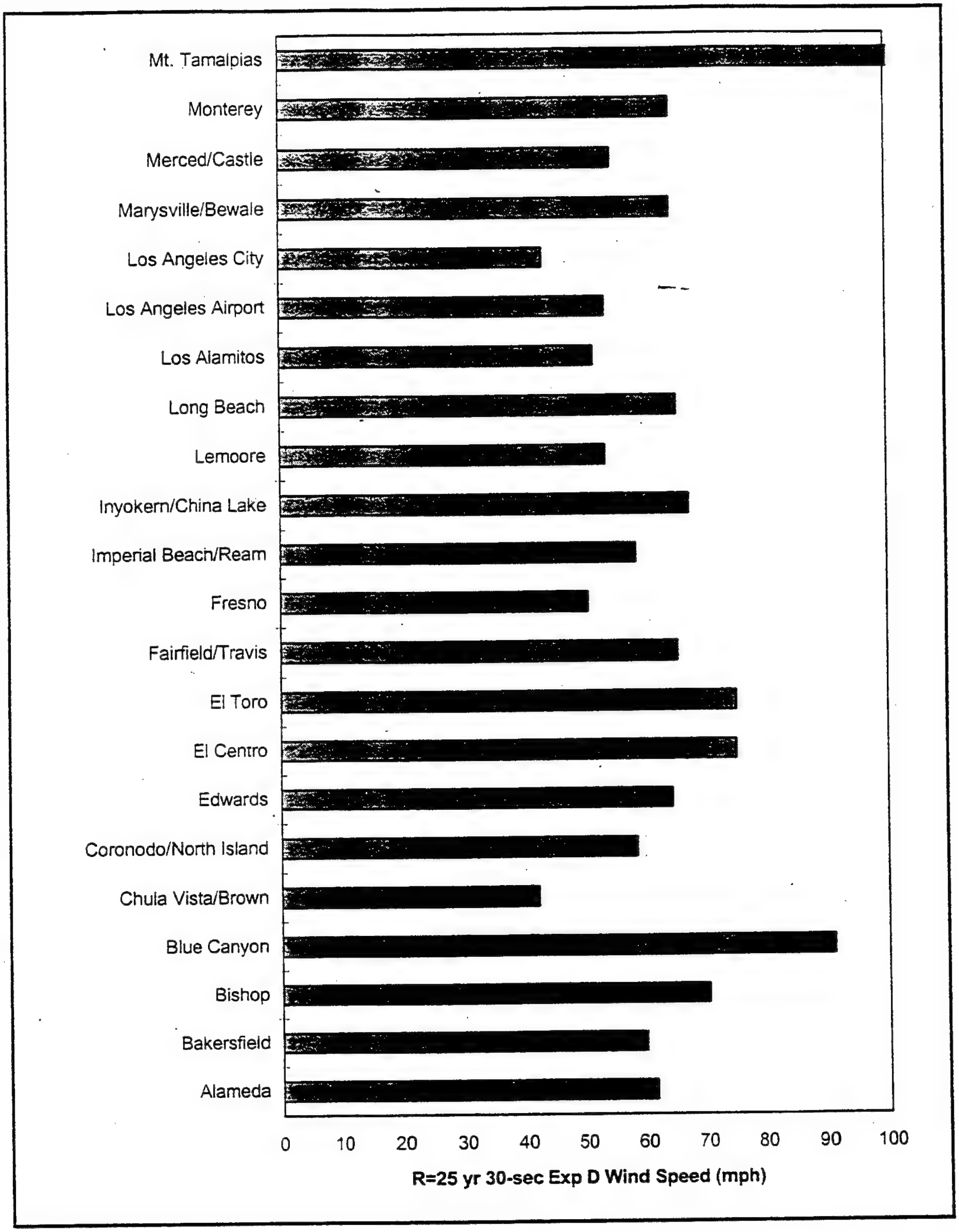


**Figure 4.1 R=25 YR 30-SEC EXP D DESIGN WIND SPEEDS**

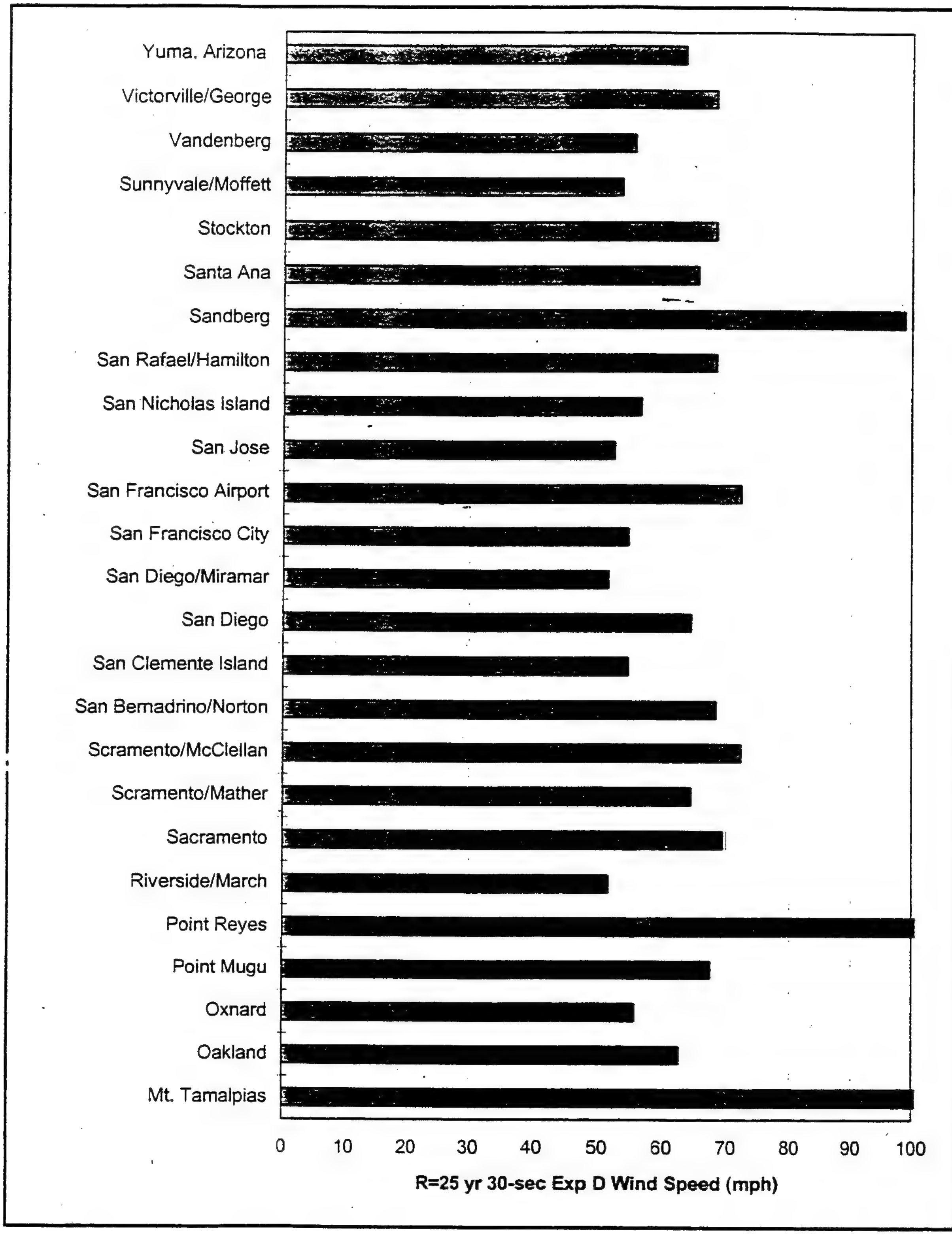


**Figure 4.2 R=25 YR 30-SEC EXP D DESIGN WIND SPEEDS CONT.**

## 5.0 SUMMARY AND RECOMMENDATIONS

The U.S. Navy is extremely interested in safely mooring ships. Therefore MIL-HDBK-1026/4 (draft of 1998) was recently funded. It is designed to be a comprehensive guide for design and inspection of mooring facilities. Many references were consulted in developing this manual. This manual was designed for all classes of ships, including tankers. The State of California may want to consider adopting or incorporating this manual into their criteria.

Mooring analyses tools, a U.S. Navy ships' database, a climate database and a facilities database are being designed to work with MIL-HDBK-1026/4. This will allow the user to quickly and easily perform computations. The State of California may wish to participate in development of these items.

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# Section 3: BASIC DESIGN PROCEDURE

3.1 Design Approach. Begin the design with specified parameters and use engineering principles to complete the design. Types of parameters associated with mooring projects are summarized in Table 3. The basic approach to performing mooring design with the ship known is given in Table 4.

Table 3
Parameters in a Mooring Project

| PARAMETER | EXAMPLES |
|---|---|
| 1. Operational Parameters | Required ship position, amount of motion allowed |
| 2. Ship Configuration | Basic ship parameters, such as length, width, draft, displacement, wind areas, mooring fitting locations, wind/current force, and moment coefficients |
| 3. Facility Configuration | Facility location, water depth, dimensions, locations/type/capacity of mooring fittings/fenders, facility condition, facility overall capacity |
| 4. Environmental Parameters | Wind speed, current speed and direction, water levels, wave conditions and possibility of ice |
| 5. Mooring Configuration | Number/size/type/location of tension members, fenders, camels, etc. |
| 6. Material Properties | Stretch/strain characteristics of the mooring tension and compression members |

Table 4
Basic Mooring Design Approach with Known Facility for a Specific Site and a Specific Ship

| STEP | NOTES |
|---|---|
| Define customer(s) requirements | Define the ship(s) to be moored, the type of service required, the maximum allowable ship motions, and situations under which the ship will leave. |
| Determine planning requirements | Define the impact/interaction with other facilities and operations, evaluate explosive arcs, determine permit requirements, establish how the mooring is to be used, review the budget and schedule. |
| Define site and environmental parameters | Determine the water depth(s), engineering soil parameters, design winds, design currents, design waves, design water levels, and evaluate access. |
| Ship characteristics | Find the engineering characteristics of the ship(s) including sail areas, drafts, displacements, ship mooring fittings, allowable hull pressures, and other parameters. |
| Ship forces/moments | Determine the forces, moments, and other key behaviors of the ship(s). |
| Evaluate mooring alternatives | Evaluate the alternatives in terms of safety, risk, cost, constructability, availability of hardware, impact on the site, watch circle, compatibility, maintenance, inspectability, and other important aspects. |
| Design Calculations | Perform static and/or dynamic analyses (if required) for mooring performance, anchor design, fender design, etc |

Table 4
Basic Mooring Design Approach with Known Facility for a Specific Site and a Specific Ship (Continued)

| STEP | NOTE |
|---|---|
| Plans/Specs | Prepare plans, specifications, and cost estimates. |
| Permits | Prepare any required environmental studies and obtain required permits. |
| Installation planning | Prepare instructions for installation, including safety and environmental protection plans. |
| Installation monitoring | Perform engineering monitoring of the installation process. |
| Testing | Perform pull tests of all anchors in mooring facilities to ensure that they hold the required load. |
| Documentation | Document the design and as-built conditions with drawings and reports. |
| Instructions | Provide diagrams and instructions to show the customer how to use and inspect the mooring. |
| Inspection | Perform periodic inspection/testing of the mooring to assure it continues to meet the customer(s) requirements. |
| Maintenance | Perform maintenance as required and document on as-built drawings. |

3.2 General Design Criteria. General design issues shown in Table 5 should be addressed during design to help ensure projects meet customers' needs.

Table 5
Design Issues

| CRITERIA | NOTES |
|---|---|
| Vessel operating conditions | Under what conditions will the vessel(s) exit? What are the operating mission requirements for the ship? What is the maximum allowable hull pressure? |
| Allowable motions | How much ship motion in the six degrees-of-freedom will be allowable for the moored ship? This is related to brow positions and use, utilities, ship loading and unloading operations, and other requirements. Note that most ships have a very high buoyancy force and moorings should be designed to allow for water level changes at a site. |
| User skills | Is the user trained and experienced in using the proposed system? What is the risk that the mooring would be improperly used? Can a design be formulated for easy and reliable use? |
| Flexibility | How flexible is the design? Can it provide for new mission requirements not yet envisioned? Can it be used with existing facilities/ships? |
| Constructability | Does the design specify readily available commercial products and is it able to be installed and/or constructed using standard techniques, tolerances, etc.? |
| Cost | Are initial and life cycle costs minimized? |
| Inspection | Can the mooring system be readily inspected to ensure continued good working condition? |
| Maintenance | Can the system be maintained in a cost-effective manner? |
| Special requirements | What special requirements does the customer have? Are there any portions of the ship that cannot come in contact with mooring elements (e.g., submarine hulls)? |

3.2.1 Mooring Service Types. There are several types of standard services that moorings provide for DOD vessels in harbors. Therefore, the facilities and ship's mooring hardware should accommodate the types of services shown in Table 6.

Table 6
Mooring Service Types

| MOORING SERVICE TYPE | DESCRIPTION |
|---|---|
| TYPE I | This category covers moorings that are used in winds of less than 34 knots and currents less than 2 knots. Moorings include ammunition facilities, fueling facilities, deperming facilities, and ports of call. Use of these moorings is normally selected concomitant with forecasted weather. |
| TYPE II | This category covers moorings that for general purpose berthing by a vessel that **will leave** prior to an approaching tropical hurricane, typhoon, or flood. |
| TYPE III | This category covers moorings that are used for **up to 2 years** by a vessel that **will not leave** prior to an approaching tropical hurricane or typhoon. Moorings include fitting-out, repair, drydocking, and overhaul berthing facilities. Ships experience this service approximately every 5 years. Facilities providing this service are nearly always occupied. |
| TYPE IV | This category covers moorings that are used for **2 years or more** by a vessel that **will not leave** in case of a hurricane, typhoon, or flood. Moorings include inactive, drydock, ship museum, and training berthing facilities. |

3.2.2 Facility Design Criteria for Mooring Service Types. Mooring facilities should be designed using the site specific criteria given in Table 7. Table 7 gives design criteria in terms of environmental design return intervals, R, and in terms of probability of exceedence, P, for 1 year of service life, N=1.

3.2.3 Ship Hardware Design Criteria for Mooring Service Types. Ship mooring hardware needs to be designed to accommodate various modes of ship operation. During Type II operation, a ship may be moored in relatively high broadside current and get caught by a sudden storm, such as a thunderstorm. Type III mooring during repair may provide the greatest potential of risk, because the ship is moored for a significant time and cannot get underway. During Type IV mooring, the ship should be aligned with the current, extra padeyes can be welded to the ship hull for mooring, etc., so special provisions can be made for long-term storage. There are several U.S. shipyards where DOD ships can undergo major repairs. The area near Norfolk/Portsmouth, Virginia has the most extreme design criteria, so use conditions derived from that site for the ship's hardware design. Bremerton, Washington, and Pearl Harbor, Hawaii have major U.S. Navy repair shipyards with lower design winds and currents at those sites. Ship mooring hardware environmental design criteria are given in Table 8.

3.2.4 Strength. Moorings should be designed and constructed to safely resist the nominal loads in load combinations defined herein without exceeding the appropriate allowable stresses for the mooring components. Normal wear of materials and inspection methods and frequency need to be considered. Due to the probable chance of simultaneous maximum occurrences of variable loads, no reduction factors should be used.

3.2.5 Serviceability. Moorings should be designed to have adequate stiffness to limit deflections, vibration, or any other deformations that adversely affect the intended use and performance of the mooring. At the same time moorings need to be flexible enough to provide for load sharing and allow for events, such as tidal changes.

Table 7
Facility Design Criteria for Mooring Service Types

| MOORING SERVICE TYPE | WIND* | CURRENT** | WATER LEVEL | WAVES |
|---|---|---|---|---|
| TYPE I | Less than 34 knots | 2 knots or less | mean lower low to mean higher high | P=1 or R=1 yr |
| TYPE II | P=0.04 (min.) R=25 yr (min.) $V_w$=64 knots (max.) | P=0.04 R=25 yr | extreme lower low to mean higher high | P=1 or R=1 yr |
| TYPE III | P=0.02 or R=50 yr | P=0.02 or R=50 yr | extreme lower low to high | P=0.02 or R=50 yr |
| TYPE IV | P=0.01 or R=100 yr | P=0.01 or R=100 yr | extreme water levels | P=0.01 or R=100 yr |

*Use exposure D (American Society of Civil Engineers (ASCE) 7-95, Minimum Design Loads for Buildings and Other Structures; flat, unobstructed area exposed to wind flowing over open water for a distance of at least 1 mile or 1.61 km) for determining design wind speeds. Note that min. = minimum return interval or probability of exceedence used for design; max. = maximum wind speed used for design.

**To define the design water depth, use T/d=0.9 for flat keeled ships; for ships with non-flat hulls, that have sonar domes or other projections, take the ship draft, T, as the mean depth of the keel and determine the water depth, d, by adding 0.61 meter (2 feet) to the maximum navigation draft of the ship.

Table 8
Ship Mooring Hardware Design Criteria

a. Ship Anchor Systems*

| MAXIMUM WATER DEPTH | MINIMUM WIND SPEED | MINIMUM CURRENT SPEED | CHAIN FACTOR OF SAFETY | ANCHOR HOLDING FACTOR OF SAFETY |
|---|---|---|---|---|
| 240 ft<br>73 m | 70 knots<br>36.0 m/s | 4 knots<br>2.06 m/s | 4.0 | 1.0 |

b. Submarine Anchor Systems*

| MAXIMUM WATER DEPTH | MINIMUM WIND SPEED | MINIMUM CURRENT SPEED | CHAIN FACTOR OF SAFETY | ANCHOR HOLDING FACTOR OF SAFETY |
|---|---|---|---|---|
| 90 ft<br>27.4 m | 70 knots<br>36.0 m/s | 4 knots<br>2.06 m/s | 4.0 | 1.0 |

c. Ship Mooring Systems**

| CONDITION | MINIMUM WIND SPEED | MINIMUM CURRENT SPEED | MOORING LINE FACTOR OF SAFETY |
|---|---|---|---|
| Normal weather condition | 25 knots<br>12.9 m/s | 1 knot<br>0.51 m/s | 9.0 |
| Heavy weather condition | 50 knots<br>25.7 m/s | 3 knots<br>1.54 m/s | 3.0 |

*Quasi-static design assuming wind and current are co-linear for ship and submarine anchor systems (after NAVSEA DDS-581).

**Quasi-static design assuming current is broadside and wind can approach from any direction (after NAVSEA DDS-582-1).

3.2.6 General Mooring Integrity. For multiple-member moorings, such as for a ship secured to a pier by a number of lines, the mooring system strongly relies on load sharing among several members. If one member is lost, the ship should remain moored. Therefore, a multiple member mooring design should be designed to ensure that remaining members maintain a factor of safety at least 75 percent of the intact mooring factors of safety shown in Table 9 with any one member missing.

3.2.7 Quasi-Static Safety Factors. Table 9 gives recommended minimum factors of safety for "quasi-static" design based on material reliability.

3.2.8 Allowable Ship Motions. Table 10 gives recommended operational ship motion criteria for moored vessels. Table 10(a) gives maximum wave conditions for manned and moored small craft (Permanent International Association of Navigation Congresses (PIANC), Criteria for Movements of Moored Ships in Harbors; A Practical Guide, 1995). These criteria are based on comfort of personnel on board a small boat, and are given as a function of boat length and locally generated.

Table 10(b) gives recommended motion criteria for safe working conditions for various types of vessels (PIANC, 1995).

Table 10(c) gives recommended velocity criteria and Table 10(d) and (e) give special criteria.

Table 9
Minimum Quasi-Static Factors of Safety

| COMPONENT | MINIMUM FACTOR OF SAFETY | NOTES |
|---|---|---|
| Stockless and balanced fluke anchors | 1.5 | For ultimate anchoring system holding capacity; use 1.0 for ship's anchoring* |
| High efficiency drag anchors (e.g. lightweight) | 2.0 | For ultimate anchoring system holding capacity; use 1.0 for ship's anchoring* |
| Fixed anchors (piles and plates) | 3.0 | For ultimate anchoring system holding capacity* |
| Deadweight anchors | - | Use carefully (see Naval Civil Engineering Laboratory (NCEL) <u>Handbook for Marine Geotechnical Engineering</u>, 1985) |
| Chain | 3.0<br>4.0 | For relatively straight lengths.<br>For chain around bends.<br>These factors of safety are for the new chain break strength. |
| Wire rope | 3.0 | For the new wire rope break strength. |
| Synthetic line** | 3.0 | For new line break strength. |
| Ship bitts | *** | Use American Institute of Steel Construction (AISC) code. |
| Pier bollards | *** | Use AISC and other applicable codes. |

*It is recommended that anchors be pull tested.
**Reduce the effective strength of wet nylon line by 15 percent.
*** For mooring fittings take 3 parts of the largest size of line used on the fitting; apply a load of: 3.0*(minimum line break strength)*1.3 to determine actual stresses, $\sigma_{act.}$; design fittings so $(\sigma_{act.}/ \sigma_{allow.})<1.0$, where $\sigma_{allow.}$ is the allowable stress from AISC and other applicable codes.

Table 10
Recommended Practical Motion Criteria for Moored Vessels

(a) Safe Wave Height Limits for Moored Manned Small Craft
(after PIANC, 1995)

| | Beam/Quartering Seas | | Head Seas | |
|---|---|---|---|---|
| Vessel Length (m) | Wave Period (sec) | Maximum Sign Wave Height, $H_s$ (m) | Wave Period (sec) | Maximum Sign Wave Height, $H_s$ (m) |
| 4 to 10 | <2.0 | 0.20 | <2.5 | 0.20 |
| " | 2.0-4.0 | 0.10 | 2.5-4.0 | 0.15 |
| " | >4.0 | 0.15 | >4.0 | 0.20 |
| 10-16 | <3.0 | 0.25 | <3.5 | 0.30 |
| " | 3.0-5.0 | 0.15 | 3.5-5.5 | 0.20 |
| " | >5.0 | 0.20 | >5.5 | 0.30 |
| 20 | <4.0 | 0.30 | <4.5 | 0.30 |
| " | 4.0-6.0 | 0.15 | 4.5-7.0 | 0.25 |
| " | >6.0 | 0.25 | >7.0 | 0.30 |

Table 10
Recommended Practical Motion Criteria for
Moored Vessels (Continued)

(b) Recommended Motion Criteria for Safe Working Conditions[1]
(after PIANC, 1995)

| Vessel Type | Cargo Handling Equipment | Surge (m) | Sway (m) | Heave (m) | Yaw (°) | Pitch (°) | Roll (°) |
|---|---|---|---|---|---|---|---|
| Fishing vessels 10-3000 GRT[2] | Elevator crane | 0.15 | 0.15 | - | - | - | - |
| | Lift-on/off | 1.0 | 1.0 | 0.4 | 3 | 3 | 3 |
| | Suction pump | 2.0 | 1.0 | - | - | - | - |
| Freighters & coasters <10000 DWT[3] | Ship's gear | 1.0 | 1.2 | 0.6 | 1 | 1 | 2 |
| | Quarry cranes | 1.0 | 1.2 | 0.8 | 2 | 1 | 3 |
| Ferries, Roll-On/ Roll-Off (RO/RO) | Side ramp[4] | 0.6 | 0.6 | 0.6 | 1 | 1 | 2 |
| | Dew/storm ramp | 0.8 | 0.6 | 0.8 | 1 | 1 | 4 |
| | Linkspan | 0.4 | 0.6 | 0.8 | 3 | 2 | 4 |
| | Rail ramp | 0.1 | 0.1 | 0.4 | - | 1 | 1 |
| General cargo 5000-10000 DWT | - | 2.0 | 1.5 | 1.0 | 3 | 2 | 5 |
| Container vessels | 100% efficient | 1.0 | 0.6 | 0.8 | 1 | 1 | 3 |
| | 50% efficient | 2.0 | 1.2 | 1.2 | 1.5 | 2 | 6 |
| Bulk carriers 30000-150000 DWT | Cranes | 2.0 | 1.0 | 1.0 | 2 | 2 | 6 |
| | Elevator/ bucket-wheel | 1.0 | 0.5 | 1.0 | 2 | 2 | 2 |
| | Conveyor belt | 5.0 | 2.5 | - | 3 | - | - |
| Oil tankers | Loading arms | 3.0[5] | 3.0 | - | - | - | - |
| Gas tankers | Loading arms | 2.0 | 2.0 | - | 2 | 2 | 2 |

Notes for Table 10(b):
[1]Motions refer to peak-to-peak values (except for sway, which is zero-to-peak)

[2]GRT = Gross Registered Tons expressed as internal volume of ship in units of 100 $ft^3$ (2.83 $m^3$)
[3]DWT = Dead Weight Tons, which is the total weight of the vessel and cargo expressed in long tons (1016 kg) or metric tons (1000 kg)
[4]Ramps equipped with rollers.
[5]For exposed locations, loading arms usually allow for 5.0-meter motion.

Table 10
Recommended Practical Motion Criteria
for Moored Vessels (Continued)

(c) Recommended Velocity Criteria for Safe Mooring Conditions for Fishing Vessels, Coasters, Freighters, Ferries and Ro/Ro Vessels (after PIANC, 1995)

| Ship Size (DWT) | Surge (m/s) | Sway (m/s) | Heave (m/s) | Yaw (°/s) | Pitch (°/s) | Roll (°/s) |
|---|---|---|---|---|---|---|
| 1000 | 0.6 | 0.6 | - | 2.0 | - | 2.0 |
| 2000 | 0.4 | 0.4 | - | 1.5 | - | 1.5 |
| 8000 | 0.3 | 0.3 | - | 1.0 | - | 1.0 |

(d) Special Criteria for Walkways and Rail Ramps (after PIANC, 1995)

| Parameter | Maximum Value |
|---|---|
| Vertical velocity | 0.2 m/s |
| Vertical acceleration | 0.5 $m/s^2$ |

Table 10
Recommended Practical Motion Criteria
for Moored Vessels (Continued)

(e) Special Criteria

| CONDITION | MAXIMUM VALUES | NOTES |
|---|---|---|
| Heave | - | Ships will move vertically with any long period water level change (tide, storm surge, flood, etc.). The resulting buoyancy forces may be high, so the mooring must be designed to provide for these motions due to long period water level changes. |
| Loading/unloading preposition ships | 0.6 m (2 feet) | Maximum ramp motion during loading/unloading moving wheeled vehicles. |
| Weapons loading/unloading | 0.6 m (2 feet) | Maximum motion between the crane and the object being loaded/unloaded. |

## 3.3 Design Methods

3.3.1 Quasi-Static Design. Practical experience has shown that in many situations such as for Mooring Service Types I and II, static analysis tools can be used to reliably determine mooring designs in harbors. Winds are a key forcing factor in mooring harbors. Winds can be highly dynamic in heavy weather conditions. However, practical experience has shown that for typical DOD ships, a wind speed with a duration of 30 seconds can be used, together with static tools, to develop safe mooring designs. The use of the 30-second duration wind speed with static tools and the approach shown in Table 11 is called "quasi-static" design.

Table 11
Quasi-Static Design Notes

| CRITERIA | NOTES |
|---|---|
| Wind speed | Determine for the selected return interval, R. For typical ships use the wind that has a duration of 30 seconds at an elevation of 10 m. |
| Wind direction | Assume the wind can come from any direction except in cases where wind data show extreme winds occur in a window of directions. |
| Current speed | Use conditions for the site (speed and direction). |
| Water levels | Use the range for the site. |
| Waves | Neglected. If waves are believed to be important, then dynamic analyses are recommended. |
| Factors of safety | Perform the design using quasi-static forces and moments (see Section 4), minimum factors of safety in Table 9, and design to assure that all criteria are met. |

3.3.2 Dynamic Mooring Analysis. Conditions during Mooring Service Types III and IV, and during extreme events can be highly dynamic. Unfortunately, the dynamic behavior of a moored ship in

shallow water can be highly complex, so dynamics cannot be fully documented in this handbook. An introduction to dynamics is provided in Section 8. Information on dynamics is found in: Dynamic Analysis of Moored Floating Drydocks, Headland et. al. (1989); Advanced Dynamics of Marine Structures, Hooft (1982); Hydrodynamic Analysis and Computer Simulation Applied to Ship Interaction During Maneuvering in Shallow Channels, Kizakkevariath (1989); David Taylor Research Center (DTRC), SPD-0936-01, User's Manual for the Standard Ship Motion Program, SMP81; Low Frequency Second Order Wave Exciting Forces on Floating Structures, Pinkster (1982); Mooring Dynamics Due to Wind Gust Fronts, Seelig and Headland (1998); and A Simulation Model for a Single Point Moored Tanker, Wichers (1988). Some conditions when mooring dynamics may be important to design or when specialized considerations need to be made are given in Table 12.

3.4 Risk. Risk is a concept that is often used to design facilities, because the probability of occurrence of extreme events (currents, waves, tides, storm surge, earthquakes, etc.) is strongly site dependent. Risk is used to ensure that systems are reliable, practical, and economical.

A common way to describe risk is the concept of 'return interval', which is the mean length of time between events. For example, if the wind speed with a return interval of R = 100 years is given for a site, this wind speed would be expected to occur, on the average, once every 100 years. However, since wind speeds are probabilistic, the specified 100-year wind speed might not occur at all in any 100-year period. Or, in any 100-year period the wind speed may be equal to or exceed the specified wind speed multiple times.

The probability or risk that an event will be equaled or exceeded one or more times during any given interval is determined from:

EQUATION: $$P = 100\%*(1-(1-1/R)^N) \qquad (1)$$

where

- P = probability, in percent, of an event being equaled or exceeded one or more times in a specified interval
- R = return interval (years)
- N = service life (years)

Figure 15 shows risk versus years on station for various selected values of return interval. For example, take a ship that is on station at a site for 20 years (N=20). There is a P=18.2 percent probability that an event with a return interval of R=100 years or greater will occur one or more times at a site in a 20-year interval.

Table 12
Conditions Requiring Special Analysis

| FACTOR | SPECIAL ANALYSIS REQUIRED |
|---|---|
| Wind | > 45 mph for small craft<br>> 75 mph for larger vessels |
| Wind waves | > 1.5 ft for small craft<br>> 4 ft for larger vessels |
| Wind gust fronts | Yes for SPMs |
| Current | > 3 knots |
| Ship waves and passing ship effects | Yes for special cases (see Kizakkevariath, 1989; Occasion, 1996; Weggel and Sorensen, 1984 & 1986) |
| Long waves (seiches and tidal waves or tsunamis) | Yes |
| Berthing and using mooring as a break | Yes (see MIL-HDBK-1025/1) |
| Parting tension member | May be static or dynamic |
| Ship impact or other sudden force on the ship | Yes (if directed) |
| Earthquakes (spud moored or stiff systems) | Yes |
| Explosion, landslide, impact | Yes (if directed) |
| Tornado (reference NUREG 1974) | Yes |
| Flood, sudden water level rise | Yes (if directed) |
| Ice forcing | Yes (if a factor) |
| Ship/mooring system dynamically unstable (e.g., SPM) | Yes (dynamic behavior of ships at SPMs can be especially complex) |
| Forcing period near a natural period of the mooring system | Yes; if the forcing period is from 80% to 120% of a system natural period |

Note: SPM = single point mooring

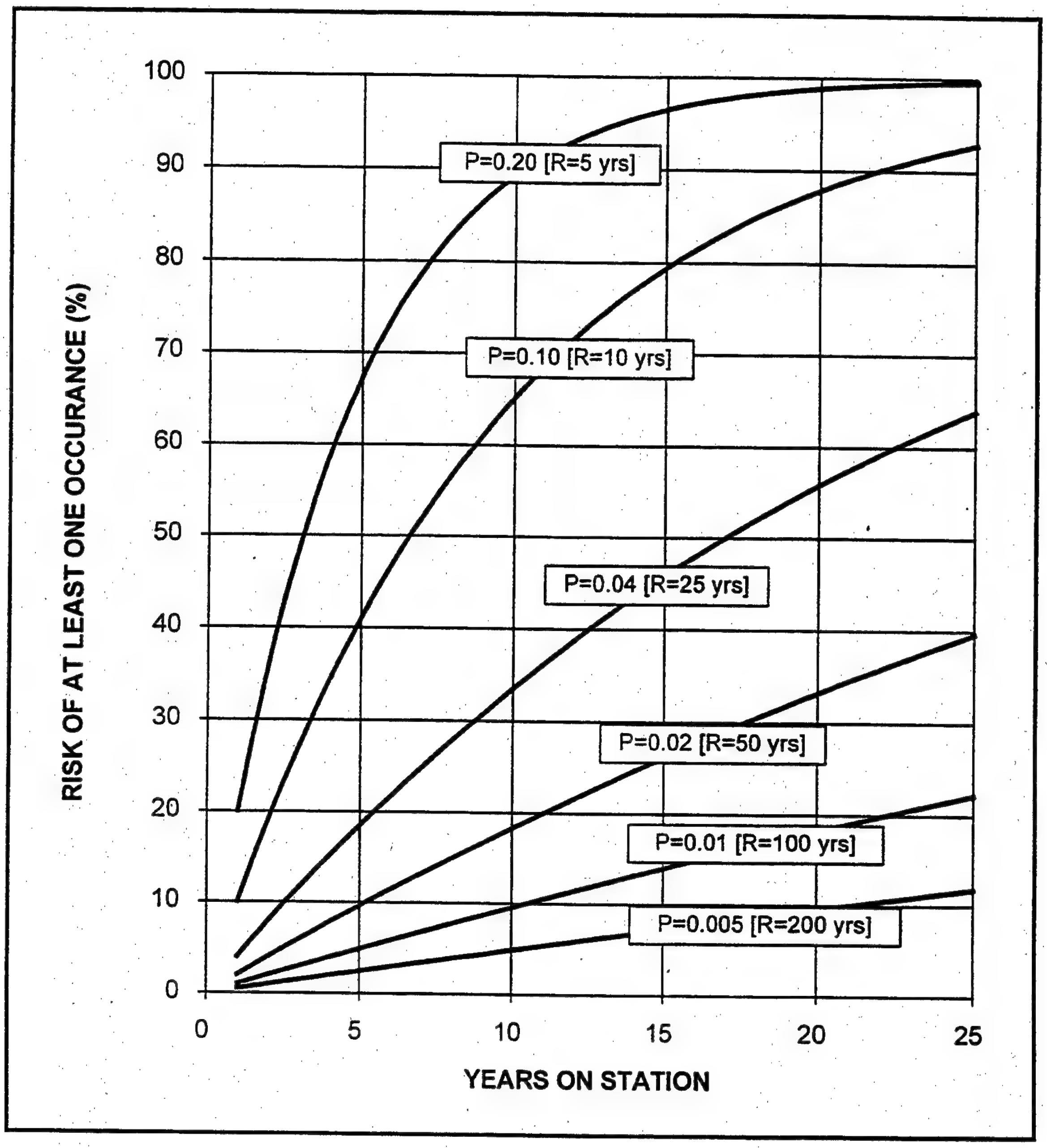



Figure 15
Risk Diagram

3.5 <u>Coordinate Systems</u>. The various coordinate systems used for ships and mooring design are described below.

3.5.1 <u>Ship Design/Construction Coordinates</u>. A forward perpendicular point (FP), aft perpendicular point (AP), and regular spaced frames along the longitudinal axes of the ship are used to define stations. The bottom of the ship keel is usually used as the reference point or "baseline" for vertical distances. Figure 16 illustrates ship design coordinates.

3.5.2 <u>Ship Hydrostatics/Hydrodynamics Coordinates</u>. The forward perpendicular is taken as Station 0, the aft perpendicular is taken as Station 20, and various cross-sections of the ship hull (perpendicular to the longitudinal axis of the ship) are used to describe the shape of the ship hull. Figure 16 illustrates ship hydrostatic conventions.

3.5.3 <u>Local Mooring Coordinate System</u>. Environmental forces on ships are a function of angle relative to the vessel's longitudinal centerline. Also, a ship tends to move about its center of gravity. Therefore, the local "right-hand-rule" coordinate system, shown in Figure 17, is used in this handbook. The midship's point is shown as a convenient reference point in Figures 17 and 18.

3.5.4 <u>Global Coordinate System</u>. Plane state grids or other systems are often used to describe x and y coordinates. The vertical datum is most often taken as relative to some water level, such as mean lower low water (MLLW).

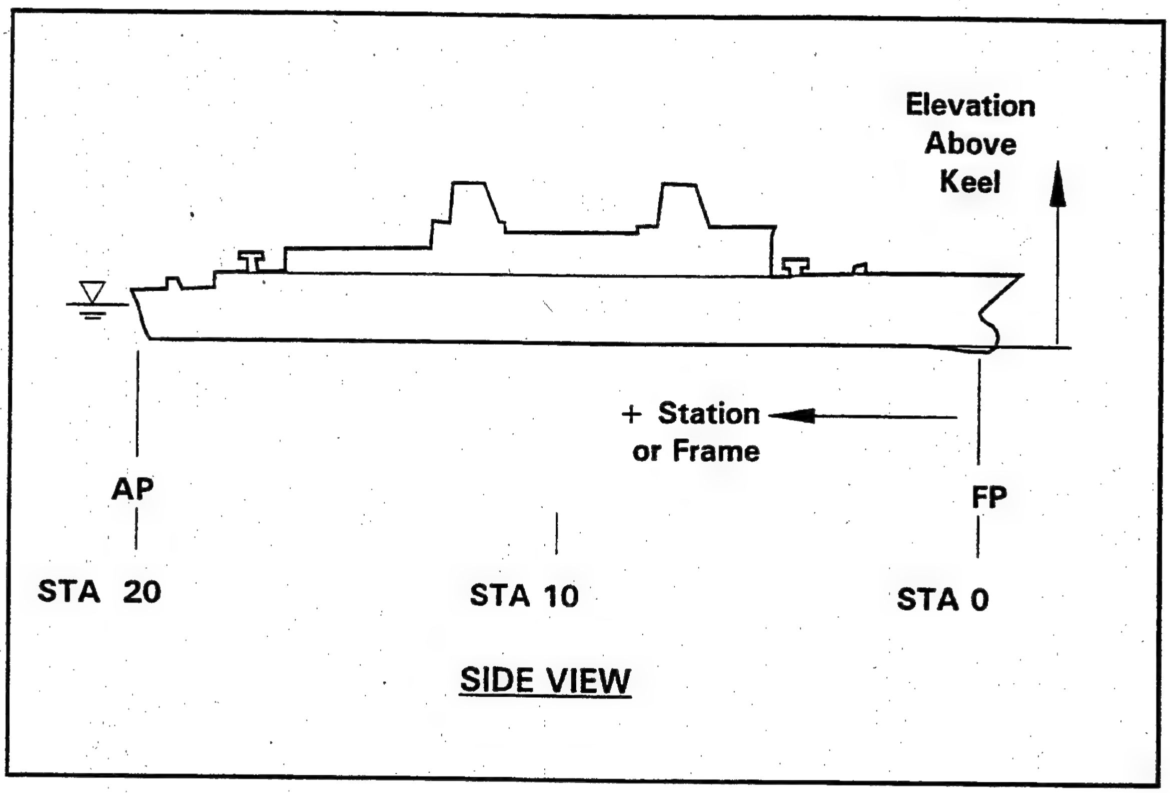



Figure 16
Ship Design and Hydrostatic Coordinates

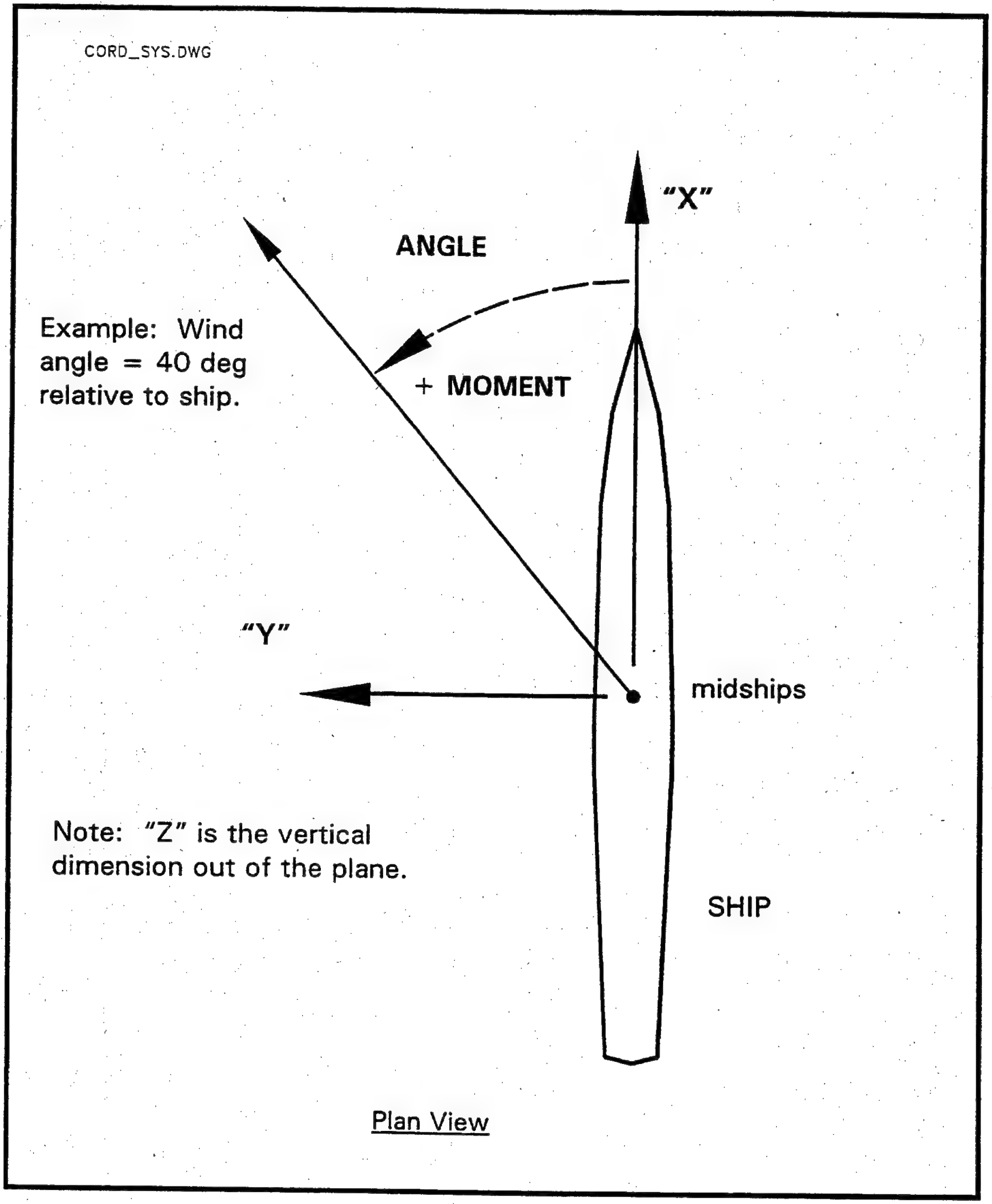
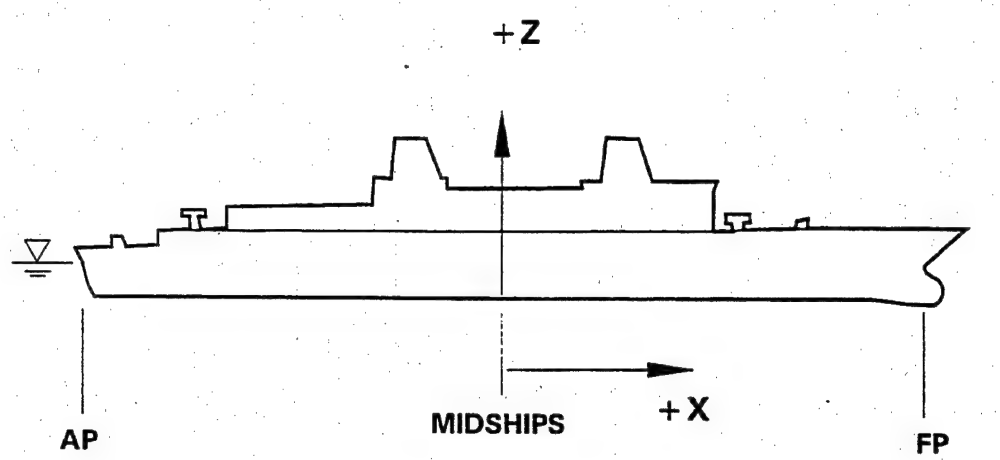



Figure 17
Local Mooring Coordinate System for a Ship




AP = Aft perpendicular
FP = Forward perpendicular
MIDSHIPS = half way between AP and FP
X = 0 at midships (LBP/2)
Y = 0 at ship centerline
Z = 0 at ship keel (baseline)

Figure 18
Local Mooring Coordinate System for a Ship

3.6 Vessel Design Considerations. Some important vessel mooring design considerations are summarized in Table 13.

Table 13
Design Considerations - Ship

| PARAMETER | NOTES |
|---|---|
| Ship fittings | The type, capacity, location, and number of mooring fittings on the ship are critical in designing moorings. |
| Ship hardware | The type, capacity, location, and number of other mooring hardware (chain, anchors, winches, etc.) on the ship are critical. |
| Buoyancy | The ship's buoyancy supports the ship up in the heave, pitch, and roll directions. Therefore, it is usually undesirable to have much mooring capacity in these directions. A large ship, for example, may have over a million pounds of buoyancy for a foot of water level rise. If an unusually large water level rise occurs for a mooring with a large component of the mooring force in the vertical direction, this could result in mooring failure. |
| Hull pressures | Ships are designed so that only a certain allowable pressure can be safely resisted. Allowable hull pressures and fender design are discussed in NFESC TR-6015-OCN, Foam-Filled Fender Design to Prevent Hull Damage. |
| Personnel access | Personnel access must be provided. |
| Hotel services | Provision must be made for utilities and other hotel services. |

3.7 Facility Design Considerations. Some important facility mooring design considerations are summarized in Table 14.

Table 14
Design Considerations - Facility

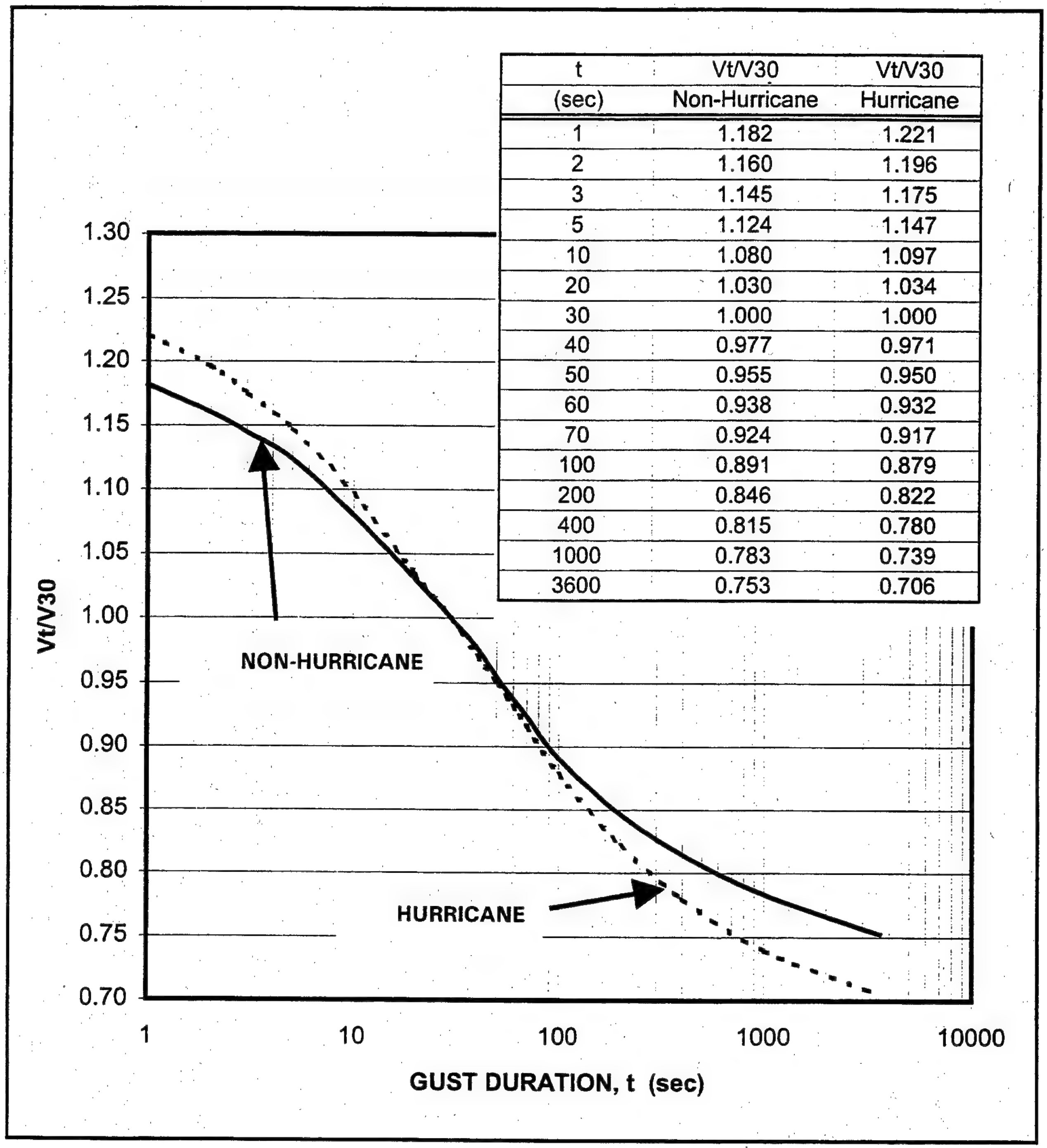
| PARAMETER | NOTES |
|---|---|
| Access | Adequate ship access in terms of channels, turning basins, bridge clearance, etc. needs to be provided. Also, tugs and pilots must be available. |
| Mooring fittings | The number, type, location and capacity of mooring fittings or attachment point have to meet the needs of all vessels using the facility. |
| Fenders | The number, type, location, and properties of marine fenders must be specified to protect the ship(s) and facility. |
| Water depth | The water depth at the mooring site must be adequate to meet the customer's needs. |
| Shoaling | Many harbor sites experience shoaling. The shoaling and possible need for dredging needs to be considered. |
| Permits | Permits (Federal, state, environmental, historical, etc.) are often required for facilities and they need to be considered. |

3.8 Environmental Forcing Design Considerations. Environmental forces acting on a moored ship(s) can be complex. Winds, currents, water levels, and waves are especially important for many designs.

3.8.1 Winds. A change in pressure from one point on the earth to another causes the wind to blow. Turbulence is carried along with the overall wind flow to produce wind gusts. If the mean wind speed and direction do not change very rapidly with time, the winds are referred to as "stationary."

Practical experience has shown that wind gusts with a duration of approximately 30 seconds or longer have a significant influence on typical moored ships with displacements of about 1000 tons or larger. Vessels with shorter natural periods can respond to shorter duration gusts. For the purposes of this handbook, a 30-second wind duration at a 10-meter (33-foot) elevation is recommended for the design for "stationary" winds. The relationship of the 30-second wind to other wind durations is shown in Figure 19.

If wind speed and/or direction changes rapidly, such as in a wind gust front, hurricane or tornado, then winds are "non-stationary". Figure 20, for example, shows a recording from typhoon OMAR in 1992 at Guam. The eye of this storm went over the recording site. The upper portion of this figure shows the wind speed and the lower portion of the figure is the wind direction. Time on the chart recorder proceeds from right to left. This hurricane had rapid changes in wind speed and direction. As the eye passes there is also a large scale change in wind speed and direction.




| t (sec) | Vt/V30 Non-Hurricane | Vt/V30 Hurricane |
|---|---|---|
| 1 | 1.182 | 1.221 |
| 2 | 1.160 | 1.196 |
| 3 | 1.145 | 1.175 |
| 5 | 1.124 | 1.147 |
| 10 | 1.080 | 1.097 |
| 20 | 1.030 | 1.034 |
| 30 | 1.000 | 1.000 |
| 40 | 0.977 | 0.971 |
| 50 | 0.955 | 0.950 |
| 60 | 0.938 | 0.932 |
| 70 | 0.924 | 0.917 |
| 100 | 0.891 | 0.879 |
| 200 | 0.846 | 0.822 |
| 400 | 0.815 | 0.780 |
| 1000 | 0.783 | 0.739 |
| 3600 | 0.753 | 0.706 |

Figure 19
Ratio of Wind Speeds for Various Gusts
(after ASCE 7-95)

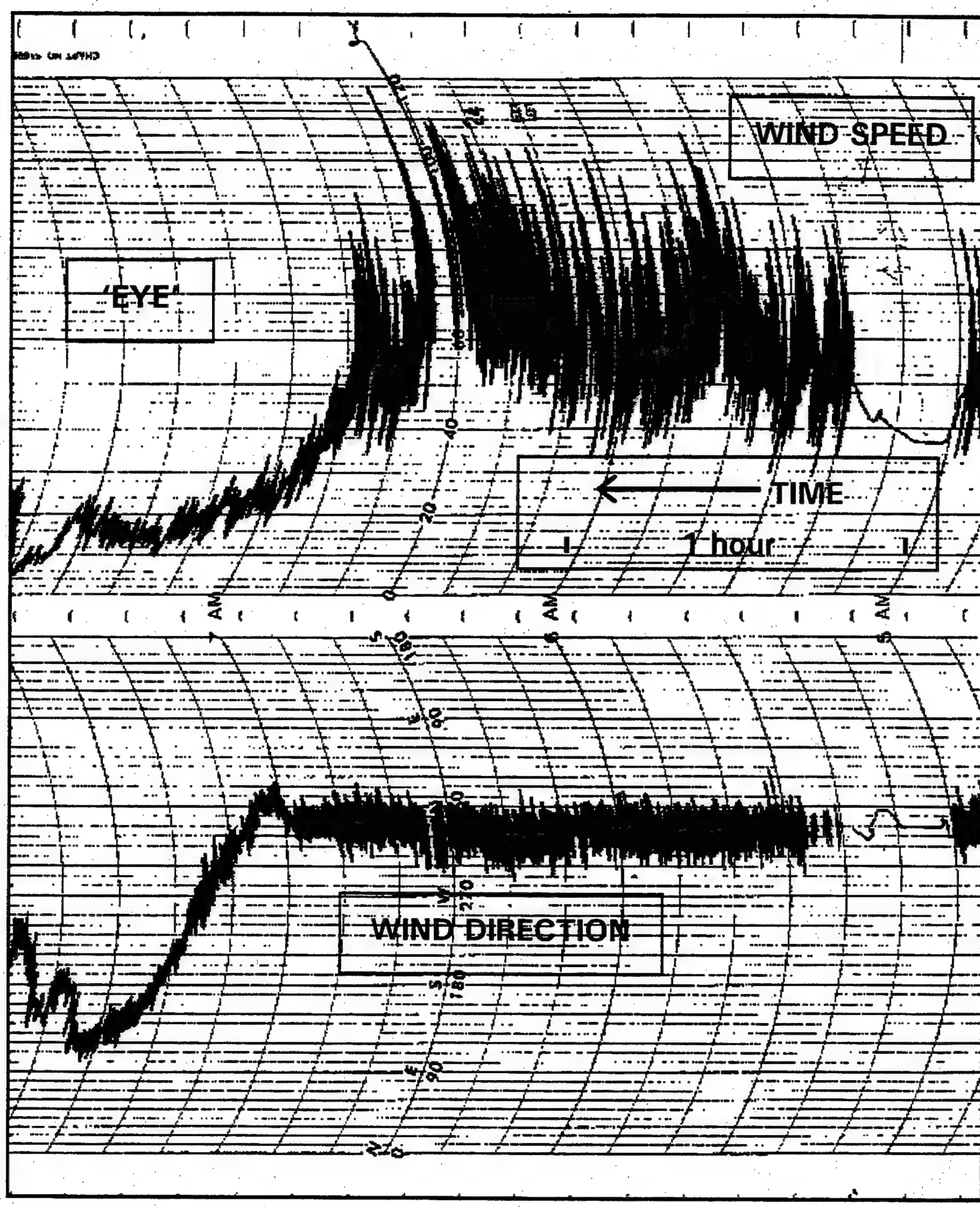



Figure 20
Typhoon *OMAR* Wind Chart Recording

3.8.2 Wind Gust Fronts. A particularly dangerous wind condition that has caused a number of mooring accidents is the wind gust front (Mooring Dynamics Due to Wind Gust Fronts, Seelig and Headland, 1998 and CHESNAVFACENGCOM, FPO-1-87(1), Failure Analysis of Hawsers on BOBO Class MSC Ships at Tinian on 7 December 1986). This is a sudden change in wind speed that is usually associated with a change in wind direction (Wind Effects on Structures, Simiu and Scanlan, 1996). The key problems with this phenomena are: (1) high mooring dynamic loads can be produced in a wind gust front, (2) there is often little warning, (3) little is known about wind gust fronts, and (4) no design criteria for these events have been established.

A study of Guam Agana National Air Station (NAS) wind records was performed to obtain some statistics of wind gust fronts (National Climatic Data Center (NCDC), Letter Report E/CC31:MJC, 1987). The 4.5 years of records analyzed from 1982 through 1986 showed approximately 500 cases of sudden wind speed change, which were associated with a shift in wind direction. These wind shifts predominately occurred in 1 minute or less and never took longer than 2 minutes to reach maximum wind speed. Figure 21 shows sudden changes in wind speed and direction that occurred over a 2-1/2 day period in October 1982. These wind gust fronts seemed to be associated with a nearby typhoon.

Table 15 gives the joint distribution of wind shifts in terms of the amount the increase in wind speed and the wind direction change. Approximately 60 percent of the wind gust fronts from 1982 through 1986 had wind direction changes in the 30-degree range, as shown in Figure 22.

Based on the Guam observations, the initial wind speed in a wind gust front ranges from 0 to 75 percent of the maximum wind speed, as shown in Figure 23. On the average, the initial wind speed was 48 percent of the maximum in the 4.5-year sample from Guam (NCDC, 1987).

Simiu and Scanlan (1996) report wind gust front increases in wind speed ranging from 3 m/sec to 30 m/sec (i.e., 6 to 60 knots). Figure 24 shows the distribution of gust front winds from the 4.5-year sample from 1982 through 1986 on Guam. This figure shows the probability of exceedence on the x-axis in a logarithmic format. The square of the wind gust front speed maximums was plotted on the y-axis, since wind force is proportional to wind speed squared. Figure 24 provides a sample of the maximum wind gust front distribution for a relatively short period at one site. Those wind gust fronts that occurred when a typhoon was nearby are identified with an "H". It can be seen that the majority of the higher gust front maximums were associated with typhoons. Also, the typhoon gust front wind speed maxima seem to follow a different distribution that the gust front maxima associated with rain and thunderstorms (see Figure 24).

Effects of winds and wind gusts are shown in the examples in Section 8 of this handbook.

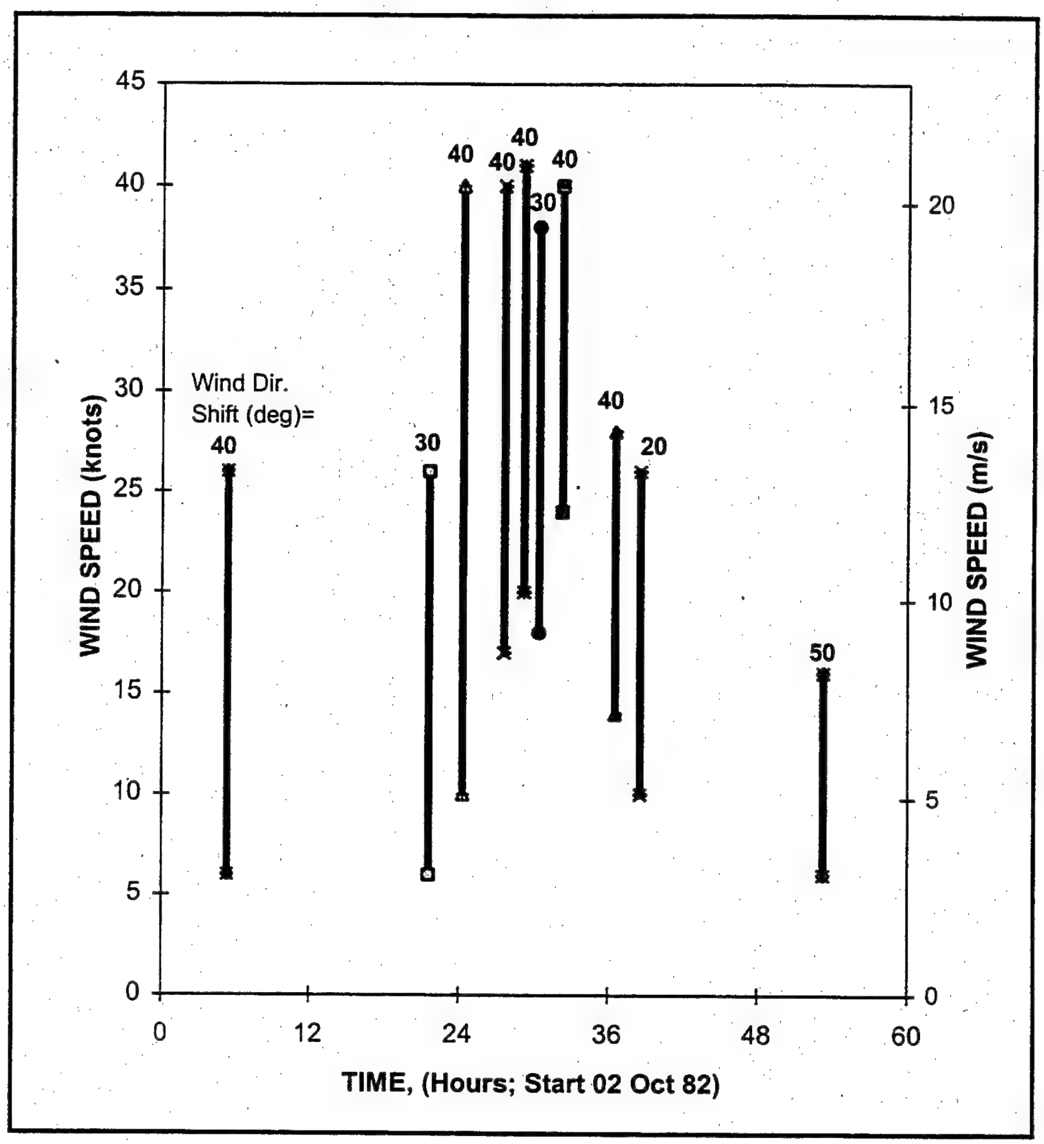



Figure 21
Sample Wind Gust Fronts on Guam, 2-4 October 1982

Table 15. Sample Distribution of Wind Gust Fronts on Guam (Agana NAS) from 1982 to 1986

| WIND SPEED CHANGE (knots) | | WIND SPEED CHANGE (m/s) | | NUMBER OF OBSERVATIONS WIND DIRECTION CHANGE | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| MIN. | MAX. | MIN. | MAX. | 20 deg | 30 deg | 40 deg | 50 deg | 60 deg | 70 deg | 80 deg | 90 deg |
| 6 | 10 | 3.1 | 5.1 | 28 | 241 | 66 | 30 | 4 | | 2 | |
| 11 | 15 | 5.7 | 7.7 | 8 | 42 | 18 | 13 | 5 | 3 | 1 | 1 |
| 16 | 20 | 8.2 | 10.3 | 6 | 7 | 3 | 2 | 2 | | | |
| 21 | 25 | 10.8 | 12.9 | | 3 | 2 | | 1 | | | |
| 26 | 30 | 13.4 | 15.4 | | | 1 | | | | | |

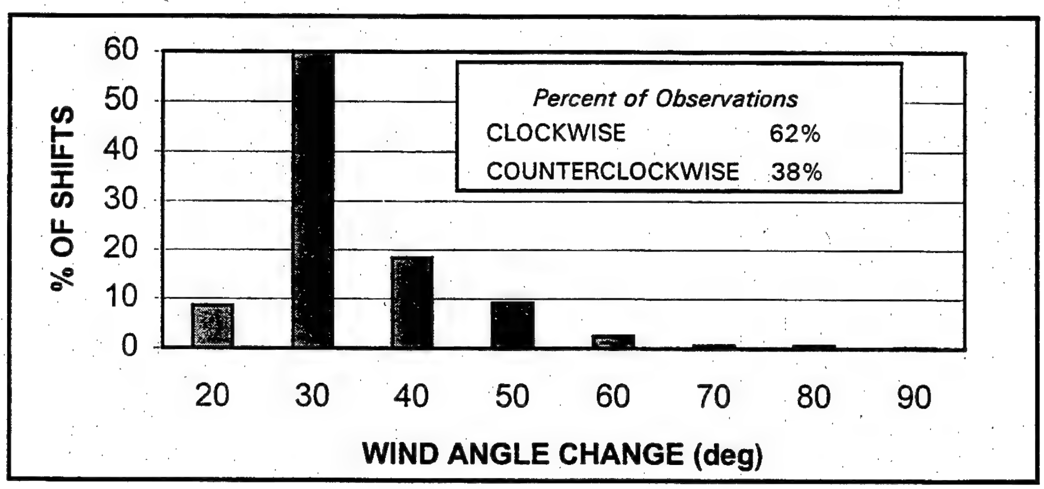



Figure 22
Distribution of Guam Wind Gust Front Wind Angle Changes

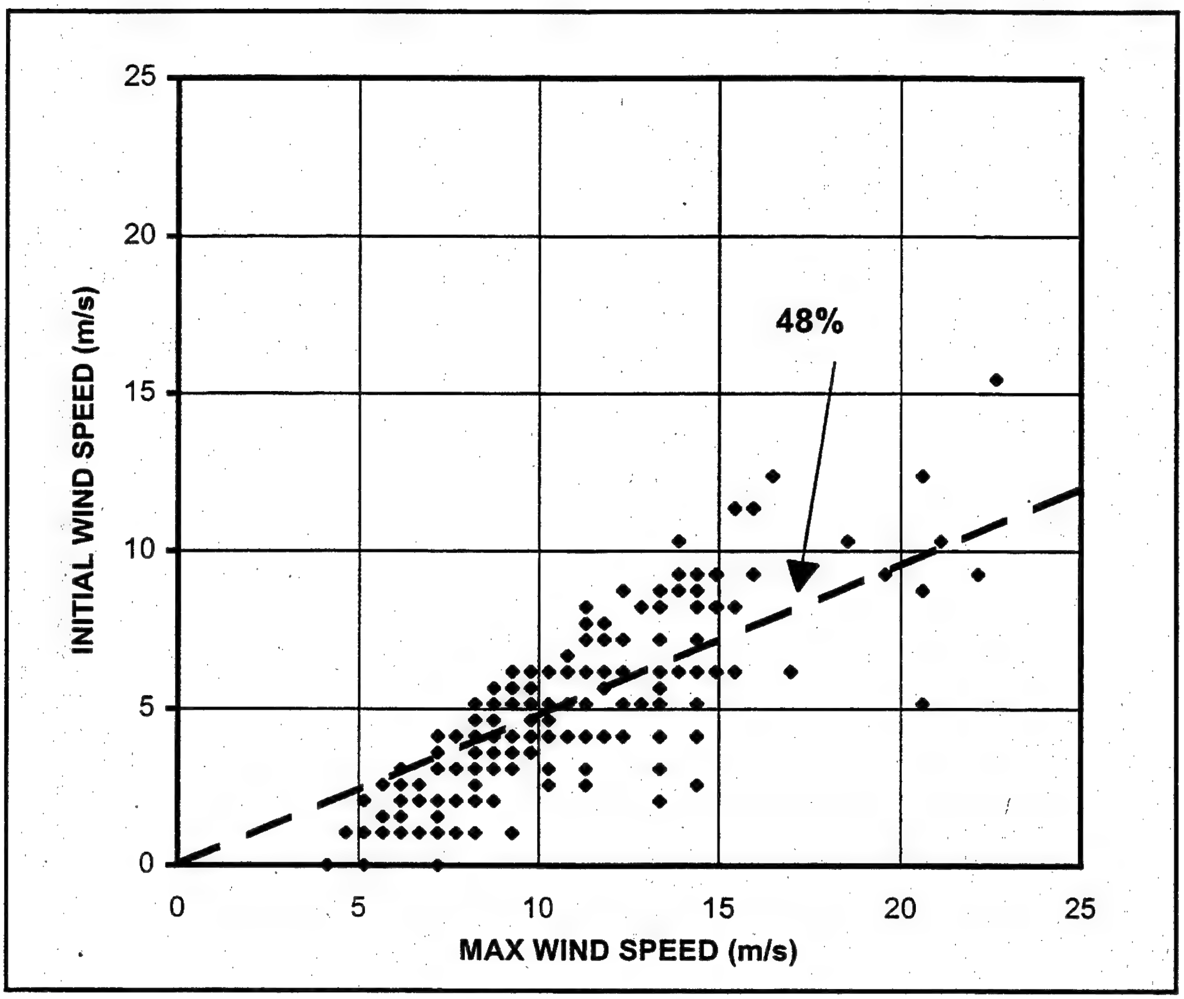



Figure 23
Initial Versus Maximum Wind Speeds for Wind Gust Fronts

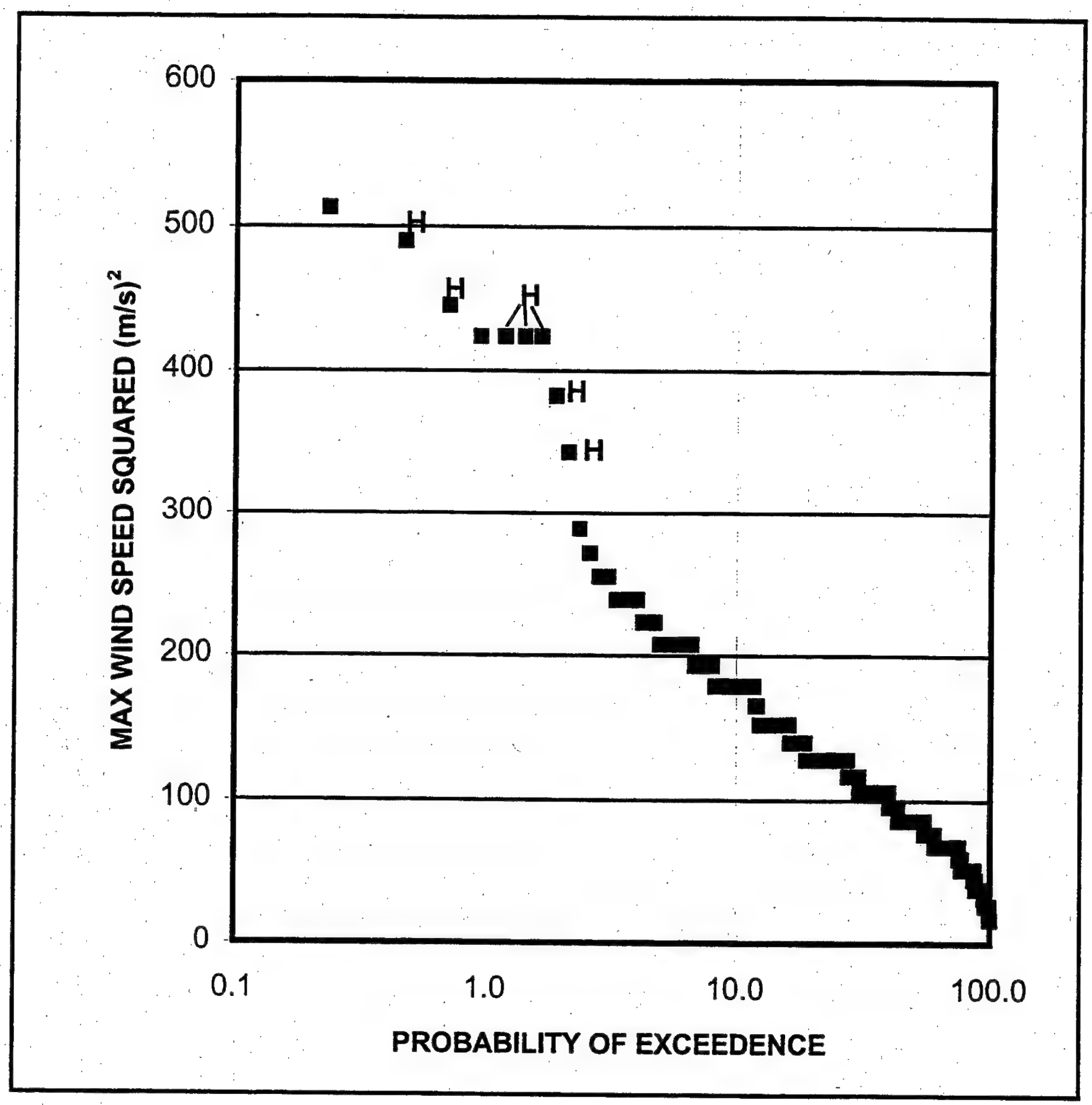



Figure 24

Wind Gust Front Maxima on Guam 1982-1986

3.8.3 Storms. Table 16 gives environmental parameters for standard storms.

Table 16
Storm Parameters

(a) Tropical Storms

| STORM | LOWER WIND SPEED (m/s) | LOWER WIND SPEED (mph) | LOWER WIND SPEED (knts) | UPPER WIND SPEED (m/s) | UPPER WIND SPEED (mph) | UPPER WIND SPEED (knts) |
|---|---|---|---|---|---|---|
| TROPICAL DEPRESSION | 10.3 | 23 | 20 | 17 | 38 | 33 |
| TROPICAL STORM | 18.0 | 40 | 35 | 32.4 | 74 | 63 |
| HURRICANE | 33.1 | 74 | 64 | - | - | - |

(b) Saffier-Simpson Hurricane Scale

| CATEGORY | WIND SPEED RANGE LOWER (m/s) | WIND SPEED RANGE LOWER (mph) | WIND SPEED RANGE UPPER (m/s) | WIND SPEED RANGE UPPER (mph) | OPEN COAST STORM SURGE RANGE LOWER (m) | OPEN COAST STORM SURGE RANGE LOWER (ft) | OPEN COAST STORM SURGE RANGE UPPER (m) | OPEN COAST STORM SURGE RANGE UPPER (ft) |
|---|---|---|---|---|---|---|---|---|
| 1 | 33.1 | 74 | 42.5 | 95 | 1.22 | 4 | 1.52 | 5 |
| 2 | 42.9 | 96 | 49.2 | 110 | 1.83 | 6 | 2.44 | 8 |
| 3 | 49.6 | 111 | 58.1 | 130 | 2.74 | 9 | 3.66 | 12 |
| 4 | 58.6 | 131 | 69.3 | 155 | 3.96 | 13 | 5.49 | 18 |
| 5 | 69.3 | 155 | - | - | 5.49 | 18 | - | - |

Table 16
Storm Parameters (Continued)

(c) Beaufort Wind Force*

| BEAUFORT WIND FORCE/ DESCRIPTION | LOWER WIND SPEED (m/s) | LOWER WIND SPEED (mph) | LOWER WIND SPEED (knts) | UPPER WIND SPEED (m/s) | UPPER WIND SPEED (mph) | UPPER WIND SPEED (knts) |
|---|---|---|---|---|---|---|
| 0 CALM | 0.0 | 0 | 0 | 0.5 | 1 | 1 |
| 1 LIGHT AIRS | 0.5 | 1 | 1 | 1.5 | 4 | 3 |
| 2 LIGHT BREEZE | 2.1 | 5 | 4 | 3.1 | 7 | 6 |
| 3 GENTLE GREEZE | 3.6 | 8 | 7 | 5.1 | 12 | 10 |
| 4 MODERATE BREEZE | 5.7 | 13 | 11 | 8.2 | 18 | 16 |
| 5 FRESH BREEZE | 8.8 | 20 | 17 | 10.8 | 24 | 21 |
| 6 STRONG BREEZE | 11.3 | 25 | 22 | 13.9 | 31 | 27 |
| 7 MODERATE GALE | 14.4 | 32 | 28 | 17.0 | 38 | 33 |
| 8 FRESH GALE | 17.5 | 39 | 34 | 20.6 | 46 | 40 |
| 9 STRONG GALE | 21.1 | 47 | 41 | 24.2 | 54 | 47 |
| 10 WHOLE GALE | 24.7 | 55 | 48 | 28.3 | 63 | 55 |
| 11 STORM | 28.8 | 65 | 56 | 32.4 | 73 | 63 |
| 12 HURRICANE | 32.9 | 74 | 64 | 36.6 | 82 | 71 |

*After <u>Handbook of Ocean and Underwater Engineers</u>, Myers et al. (1969).

Table 16
Storm Parameters (Continued)

(d) World Meteorological Organization Sea State Scale

| SEA STATE | Sign. Wave Height (ft) [m] | Sustained Wind Speed (knts) [m/s] | Modal Wave Period Range (sec) |
|---|---|---|---|
| 0 CALM/GLASSY | NONE | NONE | - |
| 1 RIPPLED | 0-0.3 [0-0.1] | 0-6 [0-3] | - |
| 2 SMOOTH | 0.3-1.6 [0.1-0.5] | 7-10 [3.6-5.1] | 3-15 |
| 3 SLIGHT | 1.6-4.1 [0.5-1.2] | 11-16 [5.7-8.2] | 3-15.5 |
| 4 MODERATE | 4.1-8.2 [1.2-2.5] | 17-21 [8.7-10.8] | 6-16 |
| 5 ROUGH | 8.2-13.1 [2.5-4.0] | 22-27 [11.3-13.9] | 7-16.5 |
| 6 VERY ROUGH | 13.1-19.7 [4.0-6.0] | 28-47 [14.4-24.2] | 9-17 |
| 7 HIGH | 19.7-29.5 [6.0-9.0] | 48-55 [24.7-28.3] | 10-18 |
| 8 VERY HIGH | 29.5-45.5[9.0-13.9] | 56-63 [28.8-32.4] | 13-19 |
| 9 PHENOMENAL | >45.5 [>13.9] | >63 [>32.4] | 18-24 |

3.8.4 Currents. The magnitude and direction of currents in harbors and nearshore areas are in most cases a function of location and time. Astronomical tides, river discharges, wind-driven currents, and other factors can influence currents. For example, wind-driven currents are surface currents that result from the stress exerted by the wind on the sea surface. Wind-driven currents generally attain a mean velocity of about 3 to 5 percent of the mean wind speed at 10 meters (33 feet) above the sea surface. The magnitude of this current strongly decreases with depth.

Currents can be very site specific, so it is recommended that currents be measured at the design site and combined with other information available to define the design current conditions.

3.8.5 Water Levels. At most sites some standard datum, such as mean low water (MLW) or mean lower low water (MLLW), is established by formal methods. Water levels are then referenced to this datum. The water level in most harbors is then a function of time. Factors influencing water levels include astronomical tides, storm surges, river discharges, winds, seiches, and other factors.

The design range in water levels at the site must be considered in the design process.

3.8.6 Waves. Most DOD moorings are wisely located in harbors to help minimize wave effects. However, waves can be important to mooring designs in some cases. The two primary wave categories of interest are:

a) Wind waves. Wind waves can be locally generated or can be wind waves or swell entering the harbor entrance(s). Small vessels are especially susceptible to wind waves.

b) Long waves. These can be due to surf beat, harbor seiching, or other effects.

Ship waves may be important in some cases. The response of a moored vessel to wave forcing includes:

a) A steady mean force.

b) First order response, where the vessel responds to each wave, and

c) Second order response, where some natural long period mode of ship/mooring motion, which usually has little damping, is forced by the group or other nature of the waves.

If any of these effects are important to a given mooring design, then a six-degree-of-freedom dynamic of the system generally needs to be considered in design. Some guidance on safe wave limits is given in Table 9

3.8.7 Water Depths. The bathymetry of a site may be complex, depending on the geology and history of dredging. Water depth may also be a function of time, if there is shoaling or scouring. Water depths are highly site specific, so hydrographic surveys of the project site are recommended.

3.8.8 Environmental Design Information. Some sources of environmental design information of interest to mooring designers are summarized in Table 17.

Table 17
Some Sources of Environmental Design Information

a. Winds

| |
|---|
| NAVFAC Climate Database, 1998 |
| ANSI/ASCE 7-95 (1996) |
| National Bureau of Standards (NBS), Series 124, Hurricane Wind Speeds in the United States, 1980 |
| Nuclear Regulatory Commission (NUREG), NUREG/CR-2639, Historical Extreme Winds for the United States - Atlantic and Gulf of Mexico Coastlines, 1982 |
| Hurricane and typhoon havens handbooks, NRL (1996) and NEPRF (1982) |
| NUREG/CR-4801, Climatology of Extreme Winds in Southern California, 1987 |
| NBS Series 118, Extreme Wind Speeds at 129 Stations in the Contiguous United States, 1979 |

b. Currents

| |
|---|
| NAVFAC Climate Database, 1998 |
| National Ocean Survey records |
| Nautical Software, Tides and Currents for Windows, 1995 |
| U.S. Army Corps of Engineers records |

Table 17
Some Sources of Environmental Design Information (Continued)

c. Water Levels

| |
|---|
| NAVFAC Climate Database, 1998 |
| Federal Emergency Management Agency records |
| U.S. Army Corps of Engineers, Special Report No. 7, Tides and Tidal Datums in the United States, 1981 |
| National Ocean Survey records |
| Hurricane and typhoon havens handbooks, NRL (1996) and NEPRF (1982) |
| Nautical Software (1995) |
| U.S. Army Corps of Engineers records |

d. Waves

| |
|---|
| Hurricane and typhoon havens handbooks, NRL (1996) and NEPRF (1982) |
| U.S. Army Corps of Engineers, Shore Protection Manual (1984) gives prediction methods |

e. Bathymetry

| |
|---|
| From other projects in the area |
| National Ocean Survey charts and surveys |
| U.S. Army Corps of Engineers dredging records |

3.9 Operational Considerations. Some important operational design considerations are summarized in Table 18.

Table 18
Mooring Operational Design Considerations

| PARAMETER | NOTES |
|---|---|
| Personnel experience/ training | What is the skill of the people using the mooring? |
| Failure | What are the consequences of failure? Are there any design features that can be incorporated that can reduce the impact? |
| Ease of use | How easy is the mooring to use and are there factors that can make it easier to use? |
| Safety | Can features be incorporated to make the mooring safer for the ship and personnel? |
| Act-of-God events | Extreme events can occur unexpectedly. Can features be incorporated to accommodate them? |
| Future use | Future customer requirements may vary from present needs. Are there things that can be done to make a mooring facility more universal? |

3.10 Inspection. Mooring systems and components should be inspected periodically to ensure they are in good working order and are safe. Table 19 gives inspection guidelines.

Table 19
Inspection Guidelines

| MOORING SYSTEM OR COMPONENT | MAXIMUM INSPECTION INTERVAL | NOTES |
|---|---|---|
| Piers and wharves | 1 year | Surface inspection |
| | 3 years | Complete inspection - wood structures |
| | 6 years | Complete inspection - concrete and steel structures |
| | | See NAVFAC MO-104.2, Specialized Underwater Waterfront Facilities Inspections; If the actual capacity/condition of mooring fittings on a pier/wharf is unknown, then pull tests are recommended to proof the fittings. |
| Fleet Moorings | 3 years | See CHESNAVFACENGCOM, FPO-1-84(6), Fleet Mooring Underwater Inspection Guidelines. Also inspect and replace anodes, if required. More frequent inspection may be required for moorings at exposed sites or for critical facilities. |
| Synthetic line | 6 months | Per manufacturer's recommendations |

Table 19
Inspection Guidelines (Continued)

| MOORING SYSTEM OR COMPONENT | MAXIMUM INSPECTION INTERVAL | NOTES |
|---|---|---|
| Ship's chain | 36 months | 0-3 years of service |
| | 24 months | 4-10 years of service |
| | 18 months | >10 years of service |
| | | (American Petroleum Institute (API) RP 2T, Recommended Practice for Planning, Designing, and Constructing Tension Leg Platforms) |
| Wire rope | 18 months | 0-2 years of service |
| | 12 months | 3-5 years of service |
| | 9 months | >5 years of service |
| | | (API RP 2T) |

3.11 Maintenance. If excessive wear or damage occurs to a mooring system, then it must be maintained. Fleet mooring chain, for example, is allowed to wear to a diameter of 90 percent of the original steel bar diameter. As measured diameters approach 90 percent, then maintenance is scheduled. Moorings with 80 to 90 percent of the original chain diameter are restricted to limited use. If a chain diameter reaches a bar diameter of 80 percent of the original diameter, then the mooring is condemned. Figure 25 illustrates some idealized models of chain wear

3.12 General Mooring Guidelines. Experience and practical considerations show that the recommendations given in Table 20 will help ensure safe mooring. These ideas apply to both ship mooring hardware and mooring facilities.

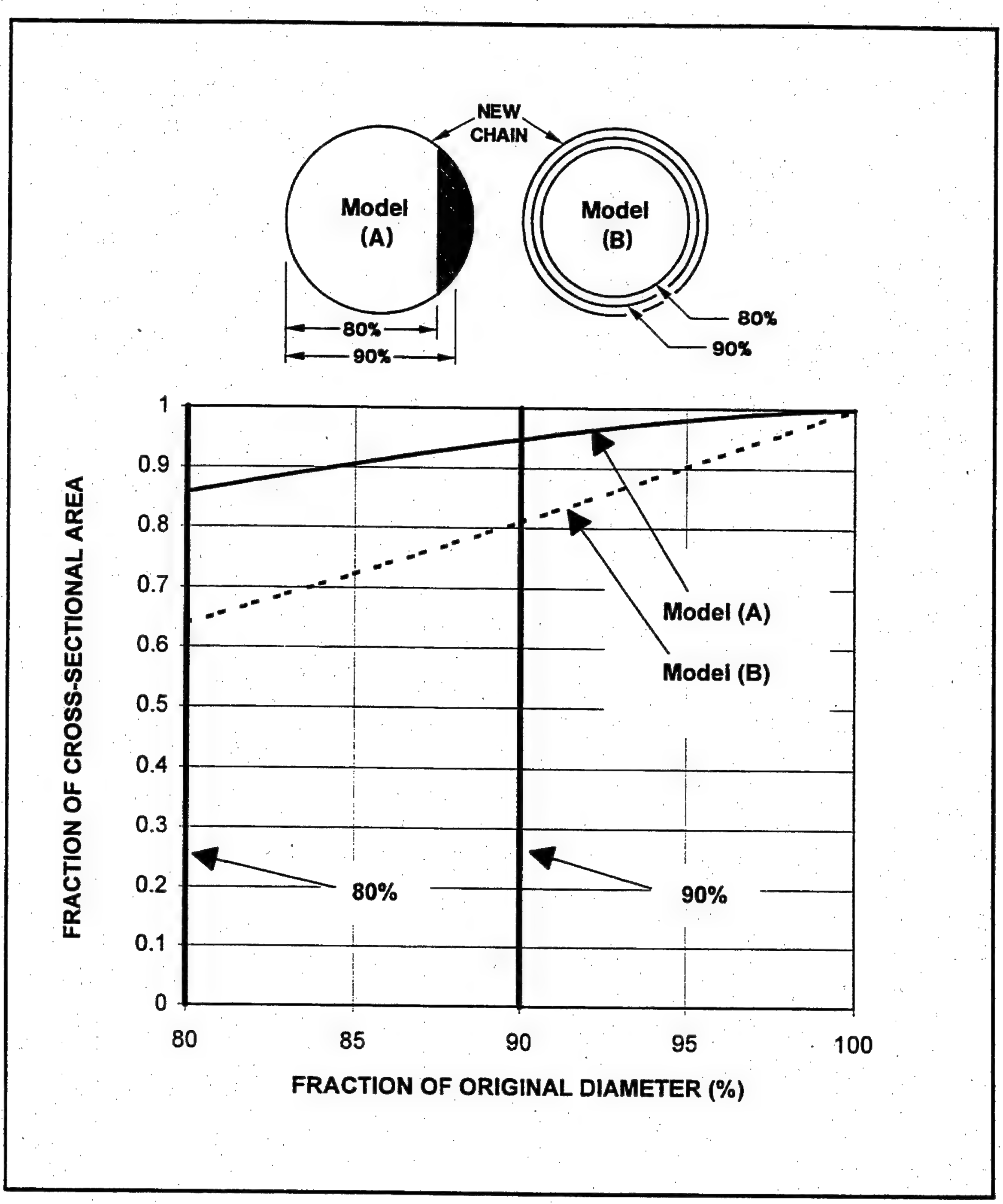



Figure 25
Idealized Models of Chain Wear

Table 20
Design Recommendations

| IDEA | NOTES |
|---|---|
| Allow ship to move with rising and falling water levels | The weight and buoyancy forces of ships can be very high, so it is most practical to design moorings to allow ships to move in the vertical direction with changing water levels. The design range of water levels for a specific site should be determined in the design process. |
| Ensure mooring system components have similar strength | A system is only as strong as its weakest segment; a system with components of similar strength can be the most economical. Mooring lines should not have a break strength greater than the capacity of the fittings they use. |
| Ensure load sharing | In some moorings, such as at a pier, many lines are involved. Ensuring that members will share the load results in the most economical system. |
| Bridle design | In cases where a ship is moored to a single point mooring buoy with a bridle, ensure that each leg of the bridle can withstand the full mooring load, because one member may take the full load as the vessel swings. |
| Provide shock absorbing in mooring systems | Wind gusts, waves, passing ships, etc., will produce transient forces on a moored ship. Allowing some motion of the ship will reduce the dynamic loads. 'Shock absorbers' including marine fenders, timber piles, synthetic lines with stretch, chain catenaries, sinkers, and similar systems are recommended to allow a moored ship to move in a controlled manner. |

Table 20
Design Recommendations (Continued)

| IDEA | NOTES |
| --- | --- |
| Limit the vertical angles of lines from ship to pier | Designing ships and piers to keep small vertical line angles has the advantages of improving line efficiency and reducing the possibility of lines pulling off pier fittings. |
| Select drag anchors to have a lower ultimate holding capacity than the breaking strength of chain and fittings | Design mooring system that uses drag anchor, so that the anchor will drag before the chain breaks. |
| Limit the loading on drag anchors to horizontal tension | Drag anchors work on the principle of 'plowing' into the soils. Keeping the mooring catenary angle small at the seafloor will aid in anchor holding. Have at least one shot of chain on the seafloor to help ensure the anchor will hold. |
| Pull test anchors whenever possible to the full design load | Pull testing anchors is recommended to ensure that all facilities with anchors provide the required holding capacity. |

# Section 4: STATIC ENVIRONMENTAL FORCES AND MOMENTS ON VESSELS

4.1 Scope. In this section design methods are presented for calculating static forces and moments on single and multiple moored vessels. Examples show calculation methods.

4.2 Engineering Properties of Water and Air. The effects of water and air at the surface of the earth are of primary interest in this section. The engineering properties of both are given in Table 21.

Table 21
Engineering Properties of Air and Water

(a) Standard Salt Water
at Sea Level at 15°C (59°F)

| PROPERTY | SI SYSTEM | ENGLISH SYSTEM |
|---|---|---|
| Mass density, $\rho_w$ | 1026 kg/m$^3$ | 1.9905 slug/ft$^3$ |
| Weight density, $\gamma_w$ | 10060 newton/m$^3$ | 64.043 lbf/ft$^3$ |
| Volume per long ton (LT) | 0.9904 m$^3$/LT | 34.977 ft$^3$/LT |
| Kinematic viscosity, $\nu$ | 1.191E-6 m$^2$/sec | 1.2817E-5 ft$^2$/sec |

(b) Standard Fresh Water
at Sea Level at 15°C (59°F)

| PROPERTY | SI SYSTEM | ENGLISH OR INCH-POUND SYSTEM |
|---|---|---|
| Mass density, $\rho_w$ | 999.0 kg/m$^3$ | 1.9384 slug/ft$^3$ |
| Weight density, $\gamma_w$ | 9797 newton/m$^3$ | 62.366 lbf/ft$^3$ |
| Volume per long ton (LT) | 1.0171 m$^3$/LT | 35.917 ft$^3$/LT |
| Volume per metric ton (ton or 1000 kg or 1 Mg) | 1.001 m$^3$/ton | 35.3497 ft$^3$/ton |
| Kinematic viscosity, $\nu$ | 1.141E-6 m$^2$/sec | 1.2285E-5 ft$^2$/sec |

Table 21
Engineering Properties of Air and Water (Continued)

(c) Air
at Sea Level at 20°C (68°F)*

| PROPERTY | SI SYSTEM | ENGLISH OR INCH-POUND SYSTEM |
|---|---|---|
| Mass density, $\rho_a$ | 1.221 kg/m$^3$ | 0.00237 slug/ft$^3$ |
| Weight density, $\gamma_a$ | 11.978 newton/m$^3$ | 0.07625 lbf/ft$^3$ |
| Kinematic viscosity, $\nu$ | 1.50E-5 m$^2$/sec | 1.615E-4 ft$^2$/sec |

* Note that humidity and even heavy rain has relatively little effect on the engineering properties of air (personal communication with the National Weather Service, 1996)

4.3 Principal Coordinate Directions. There are three primary axes for a ship:

- X - Direction parallel with the ship's longitudinal axis
- Y - Direction perpendicular to a vertical plane through the ship's longitudinal axis
- Z - Direction perpendicular to a plane formed by the "X" and "Y" axes

There are six principal coordinate directions for a ship:

- Surge - In the "X"-direction
- Sway - In the "Y"-direction
- Heave - In the "Z"-direction
- Roll - Angular about the "X"-axis
- Pitch - Angular about the "Y"-axis
- Yaw - Angular about the "Z"-axis

Of primary interest are: (1) forces in the surge and sway directions in the "X-Y" plane, and (2) moment in the yaw direction about the "Z"-axis. Ship motions occur about the center of gravity of the ship.

4.4 <u>Static Wind Forces/Moments</u>. Static wind forces and moments on stationary moored vessels are computed in this section. Figure 26 shows the definition of some of the terms used in this section. Figure 27 shows the local coordinate system.

4.4.1 <u>Static Transverse Wind Force</u>. The static transverse wind force is defined as that component of force perpendicular to the vessel centerline. In the local ship coordinate system, this is the force in the "Y" or sway direction. Transverse wind force is determined from the equation:

EQUATION: $$F_{yw} = 0.5\,\rho_a\,V_w^{\,2}\,A_y\,C_{yw}\,f_{yw}\{\theta_w\} \qquad (2)$$

where

$F_{yw}$ = transverse wind force (newtons)
$\rho_a$ = mass density of air (from Table 20)
$V_w$ = wind speed (m/s)
$A_y$ = longitudinal projected area of the ship ($m^2$)
$C_{yw}$ = transverse wind force drag coefficient
$f_{yw}\{\theta_w\}$ = shape function for transverse force
$\theta_w$ = wind angle (degrees)

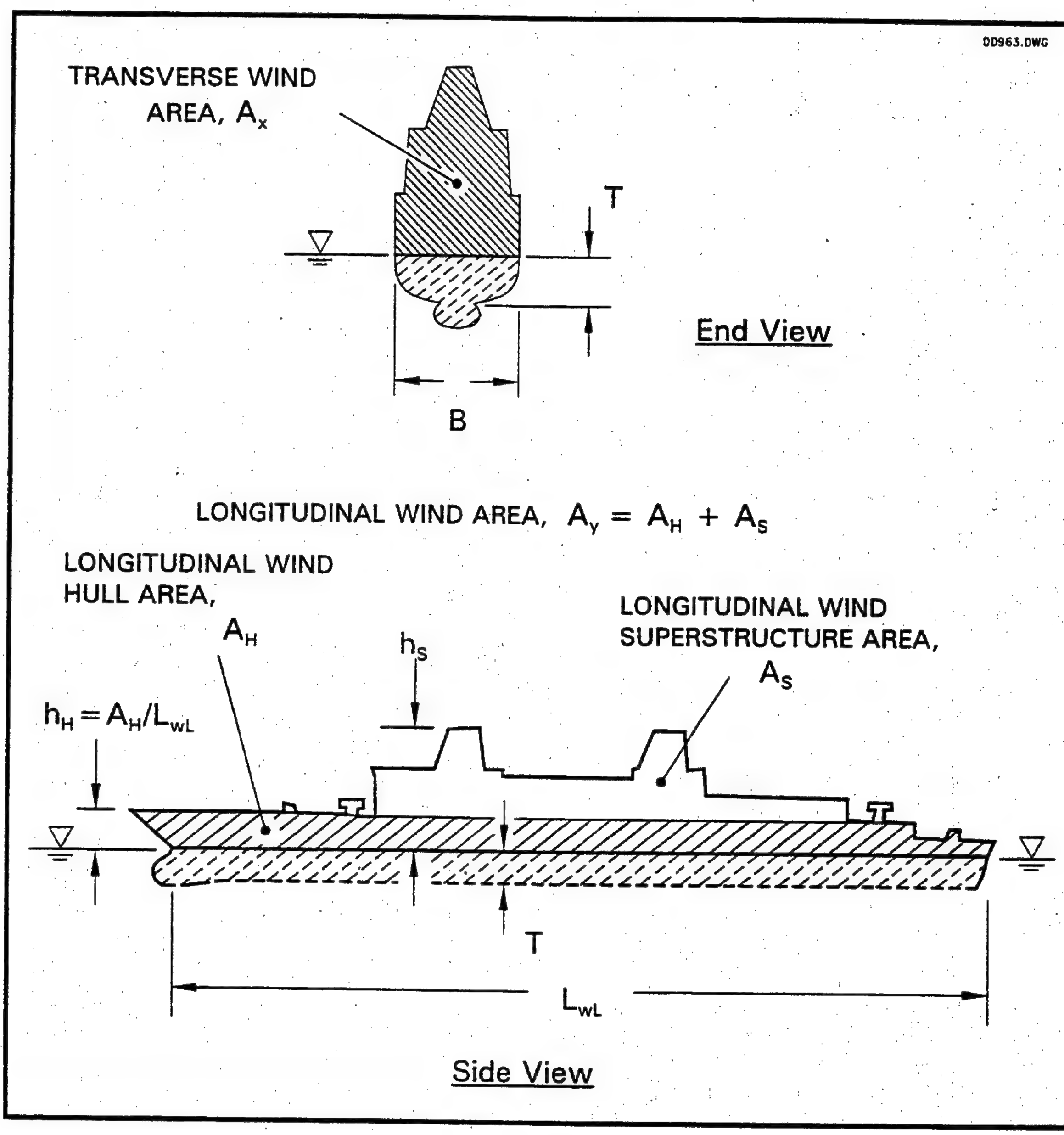



Figure 26
Definition of Terms

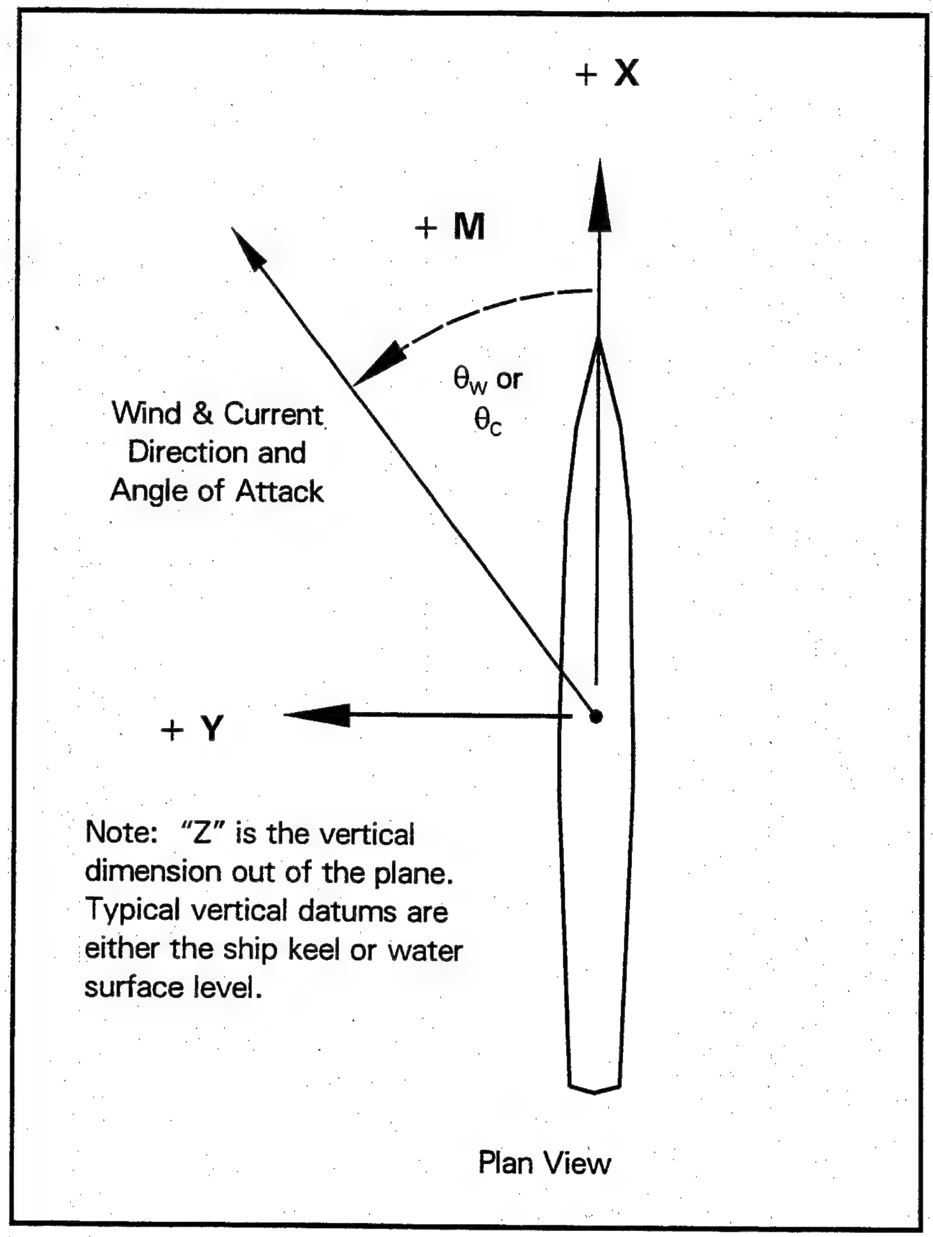



Figure 27
Local Coordinate System for a Ship

The transverse wind force drag coefficient depends upon the hull and superstructure of the vessel and is calculated using the following equation, adapted from Naval Civil Engineering Laboratory (NCEL), TN-1628, Wind-Induced Steady Loads on Ships.

EQUATION: $$C_{yw} = C*\left[((0.5(h_S + h_H))/h_R)^{2/7} A_S + (0.5*h_H/h_R)^{2/7} A_H\right]/A_Y \quad (3)$$

where

$C_{yw}$ = transverse wind force drag coefficient
$C$ = empirical coefficient, see Table 22
$h_R$ = 10 m = reference height (32.8 ft)
$h_H = A_H / L_{wL}$ = average height of the hull, defined as the longitudinal wind hull area divided by the ship length at the waterline (m)
$A_H$ = longitudinal wind area of the hull ($m^2$)
$L_{wL}$ = ship length at the waterline (m)
$h_S$ = height of the superstructure above the waterline(m)
$A_S$ = longitudinal wind area of the superstructure ($m^2$)

A recommended value for the empirical coefficient is C = 0.92 +/-0.1 based on scale model wind tunnel tests (NCEL, TN-1628). Table 22 gives typical values of C for ships and Figure 28 illustrates some ship types.

Table 22
Sample Wind Coefficients for Ships

| SHIP | C | NOTES |
|---|---|---|
| Hull dominated | 0.82 | Aircraft carriers, drydocks |
| Typical | 0.92 | ships with moderate superstructure |
| Extensive superstructure | 1.02 | Destroyers, cruisers |

The shape function for the transverse wind force (NCEL, TN-1628) is given by:

EQUATION: $$f_{yw}\{\theta_w\} = +(\sin\theta_w - 0.05*\sin\{5\theta_w\})/0.95 \qquad (4)$$

where

$f_{yw}\{\theta_w\}$ = transverse wind coefficient shape function
$\theta_w$ = wind angle (degrees)

Equation 4 is positive for wind angles $0 < \theta w < 180$ degrees and negative for wind angles $180 < \theta w < 360$ degrees. Figure 29 shows the shape and typical values for Equation 4.

These two components were derived by integrating wind over the hull and superstructure areas to obtain effective wind speeds (NCEL, TN-1628). The following example illustrates calculations of the transverse wind force drag coefficient.

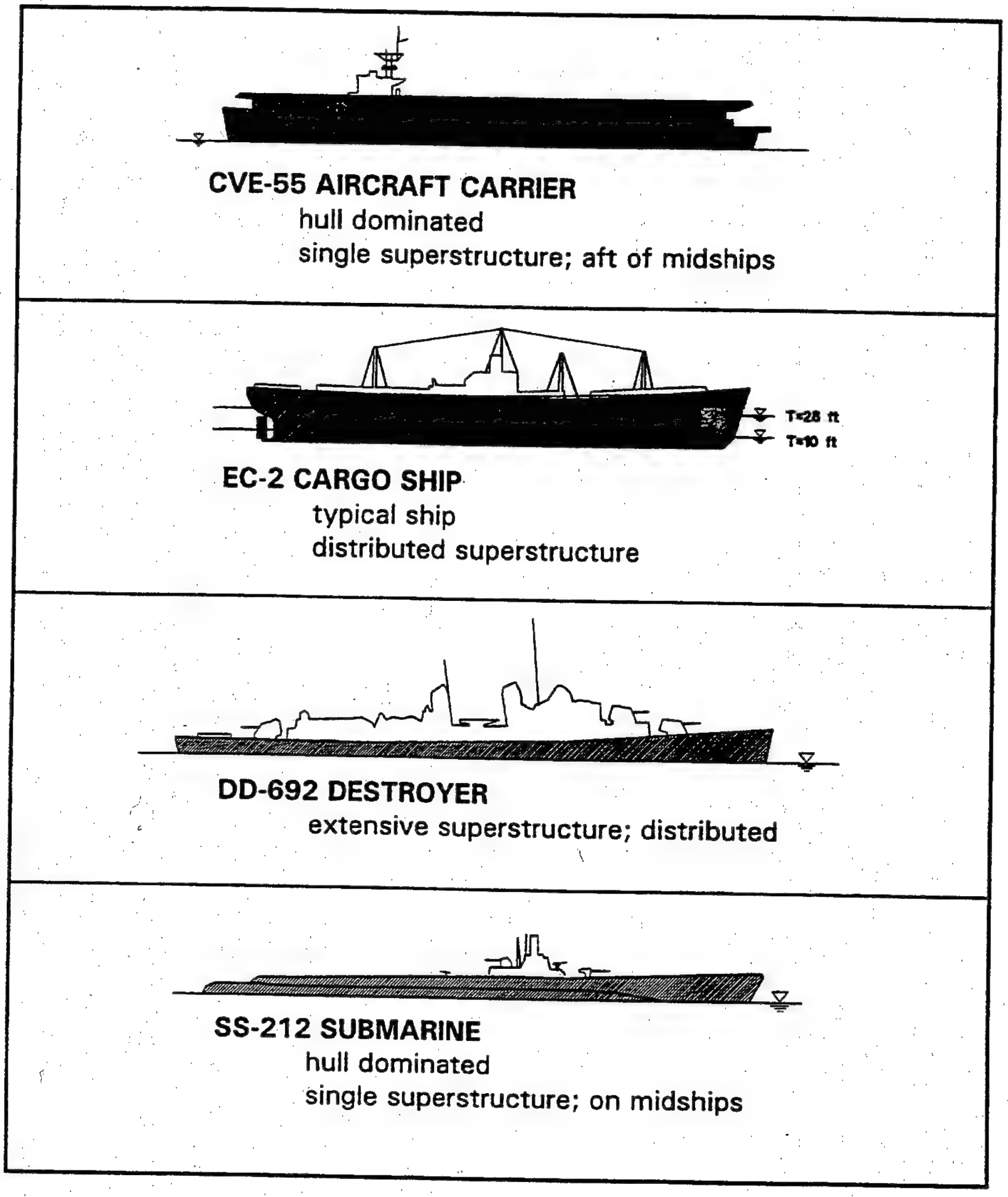



Figure 28
Sample Ship Profiles

| $\theta_w$ (deg) | $f_{wy}\{\theta_w\}$ | $\theta_w$ (deg) | $f_{wy}\{\theta_w\}$ |
|---|---|---|---|
| 0 | 0.000 | 45 | 0.782 |
| 5 | 0.069 | 50 | 0.856 |
| 10 | 0.142 | 55 | 0.915 |
| 15 | 0.222 | 60 | 0.957 |
| 20 | 0.308 | 65 | 0.984 |
| 25 | 0.402 | 70 | 0.998 |
| 30 | 0.500 | 75 | 1.003 |
| 35 | 0.599 | 80 | 1.003 |
| 40 | 0.695 | 85 | 1.001 |
| 45 | 0.782 | 90 | 1.000 |

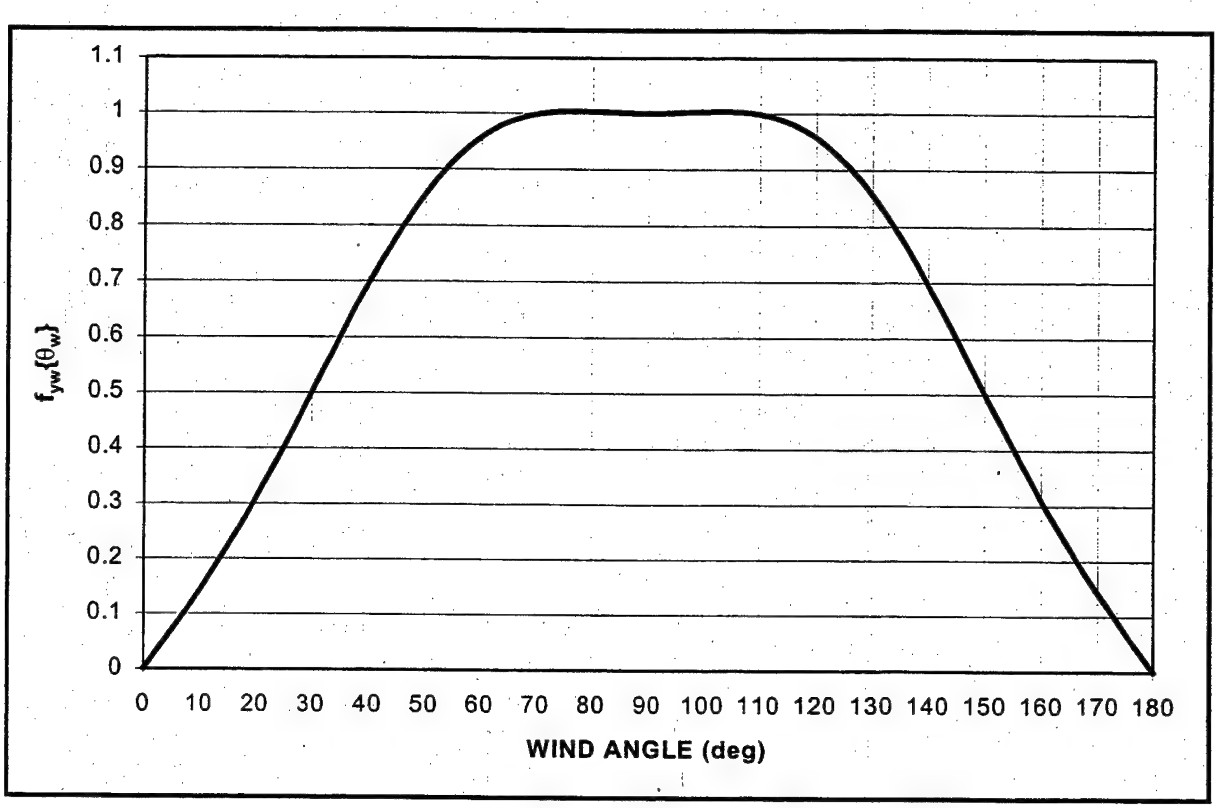



Figure 29
Shape Function for Transverse Wind Force

EXAMPLE: Find the transverse wind force drag coefficient on the destroyer shown in Figure 30.

SOLUTION: For this example the transverse wind force drag coefficient from Equation 3 is:

$$C_{yw} = C*\left[((0.5(23.9m + 6.43m))/10m)^{2/7}1203m^2 + (0.5*6.43m/10m)^{2/7}1036.1m^2\right]/2239m^2$$

$$C_{yw} = 0.940*C.$$

Destroyers have extensive superstructure, so a recommended value of C = 1.02 is used to give a transverse wind force drag coefficient of $C_{yw}$ = 0.940*1.02 = 0.958.

Note that for cases where an impermeable structure, such as a wharf, is immediately next to the moored ship, the exposed longitudinal wind area and resulting transverse wind force can be reduced. Figure 31 shows an example of a ship next to a wharf. For Case (A), wind from the water, there is no blockage in the transverse wind force and elevations of the hull and superstructure are measured from the water surface. For Case (B), wind from land, the longitudinal wind area of the hull can be reduced by the blocked amount and elevations of hull and superstructure can be measured from the wharf elevation.

Cases of multiple ships are covered in Section 4.6.

| PARAMETER | VALUE (SI UNITS) | VALUE (ENGLISH) |
|---|---|---|
| $L_{wL}$ | 161.23 m | 529 ft |
| $A_Y$ | 2239 $m^2$ | 24100 $ft^2$ |
| $A_H$ | 1036 $m^2$ | 11152 $ft^2$ |
| $A_S$ | 1203 $m^2$ | 12948 $ft^2$ |
| $h_H = A_H/L_{wL}$ | 6.43 m | 21.1 ft |
| $h_S$ | 23.9 m | 78.4 ft |

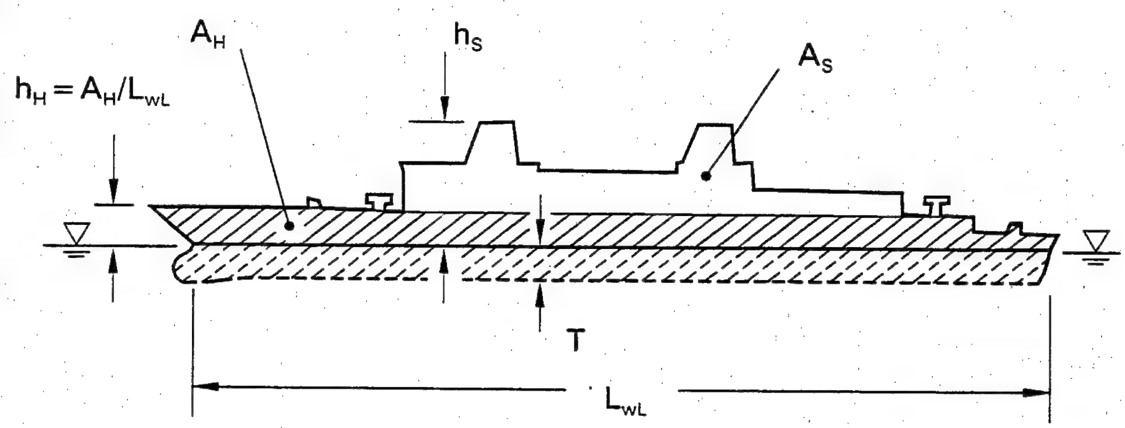



Figure 30
Example

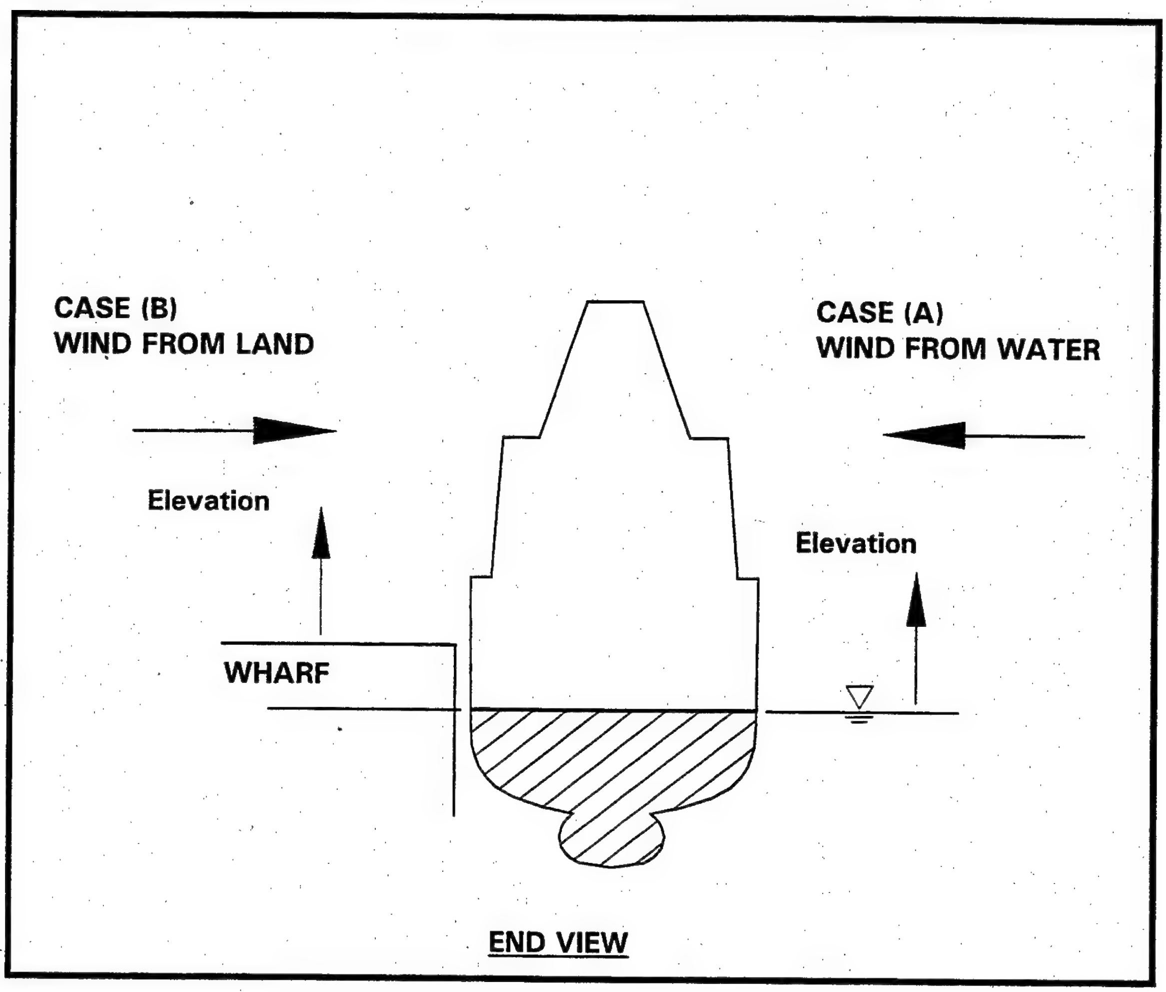



Figure 31
Blockage Effect for an Impermeable Structure
Next to a Moored Ship

4.4.2 Static Longitudinal Wind Force. The static longitudinal wind force on a vessel is defined as that component of wind force parallel to the centerline of the vessel. This is the force in the "X" or surge direction in Figure 27. Figure 26 shows the definition of winds areas.

The longitudinal force is determined from NCEL, TN-1628 using the equation:

EQUATION: $$F_{xw} = 0.5\,\rho_a\,V_w^2 A_x\,C_{xw}\,f_{xw}(\theta_w) \tag{5}$$

where

$F_{xw}$ = longitudinal wind force (newtons)
$\rho_a$ = mass density of air (from Table 21)
$V_w$ = wind speed (m/s)
$A_x$ = transverse wind area of the ship ($m^2$)
$C_{xw}$ = longitudinal wind force drag coefficient
$f_{xw}(\theta_w)$= shape function for longitudinal force
$\theta_w$ = wind angle (degrees)

The longitudinal wind force drag coefficient, $C_{xw}$, depends on specific characteristics of the vessel. Additionally, the wind force drag coefficient varies depending on bow ($C_{xwB}$) or stern ($C_{xwS}$) wind loading. Types of vessels are given in three classes: hull dominated, normal, and excessive superstructure. Recommended values of longitudinal wind force drag coefficients are given in Table 23.

Table 23
Recommended Ship Longitudinal Wind Force Drag Coefficients

| VESSEL TYPE | $C_{xwB}$ | $C_{xwS}$ |
|---|---|---|
| Hull Dominated (aircraft carriers, submarines, passenger liners) | 0.40 | 0.40 |
| Normal* | 0.70 | 0.60 |
| Center-Island Tankers* | 0.80 | 0.60 |
| Significant Superstructure (destroyers, cruisers) | 0.70 | 0.80 |

*An adjustment of up to +0.10 to $C_{xwB}$ and $C_{xwS}$ should be made to account for significant cargo or cluttered decks.

The longitudinal shape function also varies over the bow and stern wind loading regions. As the wind direction varies from headwind to tailwind, there is an angle at which the force changes sign. This is defined as $\theta_x$ and is dependent on the location of the superstructure relative to midships. Recommended values of this angle are given in Table 24.

Table 24
Recommended Values of $\theta_x$

| LOCATION OF SUPERSTRUCTURE | $\theta_x$ (deg) |
|---|---|
| Just forward of midships | 100 |
| On midships | 90 |
| Aft of midships (tankers) | 80 |
| Warships | 70 |
| Hull dominated | 60 |

Shape functions are given for general vessel categories below:

CASE I SINGLE DISTINCT SUPERSTRUCTURE

The shape function for longitudinal wind load for ships with single, distinct superstructures and hull-dominated ships is given below (examples include aircraft carriers, EC-2, and cargo vessels):

EQUATION: $$f_{xw}(\theta_w) = \cos(\phi) \tag{6}$$

where $$\phi_- = \left(\frac{90°}{\theta_x}\right)\theta_w \quad \text{for } \theta_w < \theta_x \tag{6a}$$

$$\phi_+ = \left(\frac{90°}{180°-\theta_x}\right)(\theta_w - \theta_x) + 90° \quad \text{for } \theta_w > \theta_x \tag{6b}$$

$\theta_x$ = incident wind angle that produces no net longitudinal force (Table 24)

$\theta_w$ = wind angle

Values of $f_{xw}(\theta_w)$ are symmetrical about the longitudinal axis of the vessel. So when $\theta_w > 180°$, use $360° - \theta_w$ as $\theta_w$ in determining the shape function.

CASE II DISTRIBUTED SUPERSTRUCTURE

EQUATION: $$f_{xw}(\theta_w) = \frac{\left(\sin(\gamma) - \frac{\sin(5\gamma)}{10}\right)}{0.9} \tag{7}$$

where $$\gamma_- = \left(\frac{90°}{\theta_x}\right)\theta_w + 90° \quad \text{for } \theta_w < \theta_x \tag{7a}$$

$$\gamma_+ = \left(\frac{90°}{180°-\theta_x}\right)(\theta_W) + \left(180° - \left(\frac{90°\,\theta_x}{180°-\theta_x}\right)\right) \quad \text{for } \theta_w > \theta_x \tag{7b}$$

Values of $f_{xw}(\theta_w)$ are symmetrical about the longitudinal axis of the vessel. So when $\theta_w > 180°$, use $360° - \theta_w$ as $\theta_w$ in determining the shape function. Note that the maximum longitudinal wind force for these vessels occurs for wind directions slightly off the ship's longitudinal axis.

EXAMPLE: Find the longitudinal wind drag coefficient for a wind angle of 40 degrees for the destroyer shown in Figure 30.

SOLUTION: For this destroyer, the following values are selected:

$\theta_x$ = 70° from Table 24

$C_{xwB}$ = 0.70 from Table 23

$C_{xwS}$ = 0.80 from Table 23

This ship has a distributed superstructure and the wind angle is less than the crossing value, so Equation 7a is used to determine the shape function:

$$\gamma\_ = \left(90° / (70°)\right)40° + 90° = 141.4°$$

$$f_{xw}(\theta_w) = \frac{\left(\sin(141.4°) - \frac{\sin(5*141.4°)}{10}\right)}{0.9} = 0.72$$

At the wind angle of 40 degrees, the wind has a longitudinal component on the stern. Therefore, the wind longitudinal drag coefficient for this example is:

$$C_{xw}\, f_{xw}(\theta_w) = 0.8 * 0.72 = 0.57$$

4.4.3 Static Wind Yaw Moment. The static wind yaw moment is defined as the product of the associated transverse wind force and its distance from the vessel's center of gravity. In the local ship coordinate system, this is the moment about the "Z" axis. Wind yaw moment is determined from the equation:

EQUATION: $$M_{xyw} = 0.5\,\rho_a\,V_w^2\,A_y\,L C_{xyw}\{\theta_w\} \qquad (8)$$

where

$M_{xyw}$ = wind yaw moment (newton*m)
$\rho_a$ = mass density of air (from Table 21)
$V_w$ = wind speed (m/s)
$A_y$ = longitudinal projected area of the ship ($m^2$)
L = length of ship (m)
$C_{xyw}\{\theta_w\}$ = normalized yaw moment coefficient
= moment arm divided by ship length
$\theta_w$ = wind angle (degrees)

The normalized yaw moment coefficient depends upon the vessel type. Equation 9 gives equations for computing the value of the yaw moment coefficient and Table 25 gives empirical parameter values for selected vessel types. The normalized yaw moment variables is found from:

EQUATION: $$C_{xyw}\{\theta_w\} = -a1 * \sin\left(\frac{\theta_w * 180}{\theta_z}\right) \qquad 0<\theta_w<\theta_z \qquad (9)$$

$$C_{xyw}\{\theta_w\} = a2 * \sin\left[(\theta_w - \theta_z) * \lambda\right] \qquad \theta_z \le \theta_w < 180 \text{ deg} \qquad (9a)$$

and symmetrical about the longitudinal axis of the vessel, where

$C_{xyw}\{\theta_w\}$ = normalized wind yaw moment coefficient
$a1$ = negative peak value (from Table 25)
$a2$ = positive peak value (from Table 25)
$\theta_w$ = wind angle (degrees)
$\theta_z$ = zero moment angle (degrees) (from Table 25)

$$\lambda = \frac{180*\deg}{\left[(180*\deg - \theta_z)\right]} \quad \text{(dimensionless)} \qquad (9b)$$

Table 25
Normalized Wind Yaw Moment Variables

| SHIP TYPE | Zero Moment Angle ($\theta_z$) | Negative Peak (a1) | Positive Peak (a2) | NOTES |
|---|---|---|---|---|
| Liner | 80 | 0.075 | 0.14 | |
| Carrier | 90 | 0.068 | 0.072 | |
| Tanker | 95 | 0.077 | 0.07 | Center island w/ cluttered deck |
| Tanker | 100 | 0.085 | 0.04 | Center island w/ trim deck |
| Cruiser | 90 | 0.064 | 0.05 | |
| Destroyer | 68 | 0.02 | 0.12 | |
| Others: | 130 | 0.13 | 0.025 | stern superstructure |
| | 102 | 0.096 | 0.029 | aft midships superstructure |
| | 90 | 0.1 | 0.1 | midships superstructure |
| | 75 | 0.03 | 0.05 | forward midships superstructure |
| | 105 | 0.18 | 0.12 | bow superstructure |

A plot of the yaw normalized moment coefficient for the example shown in Figure 30 is given as Figure 32.

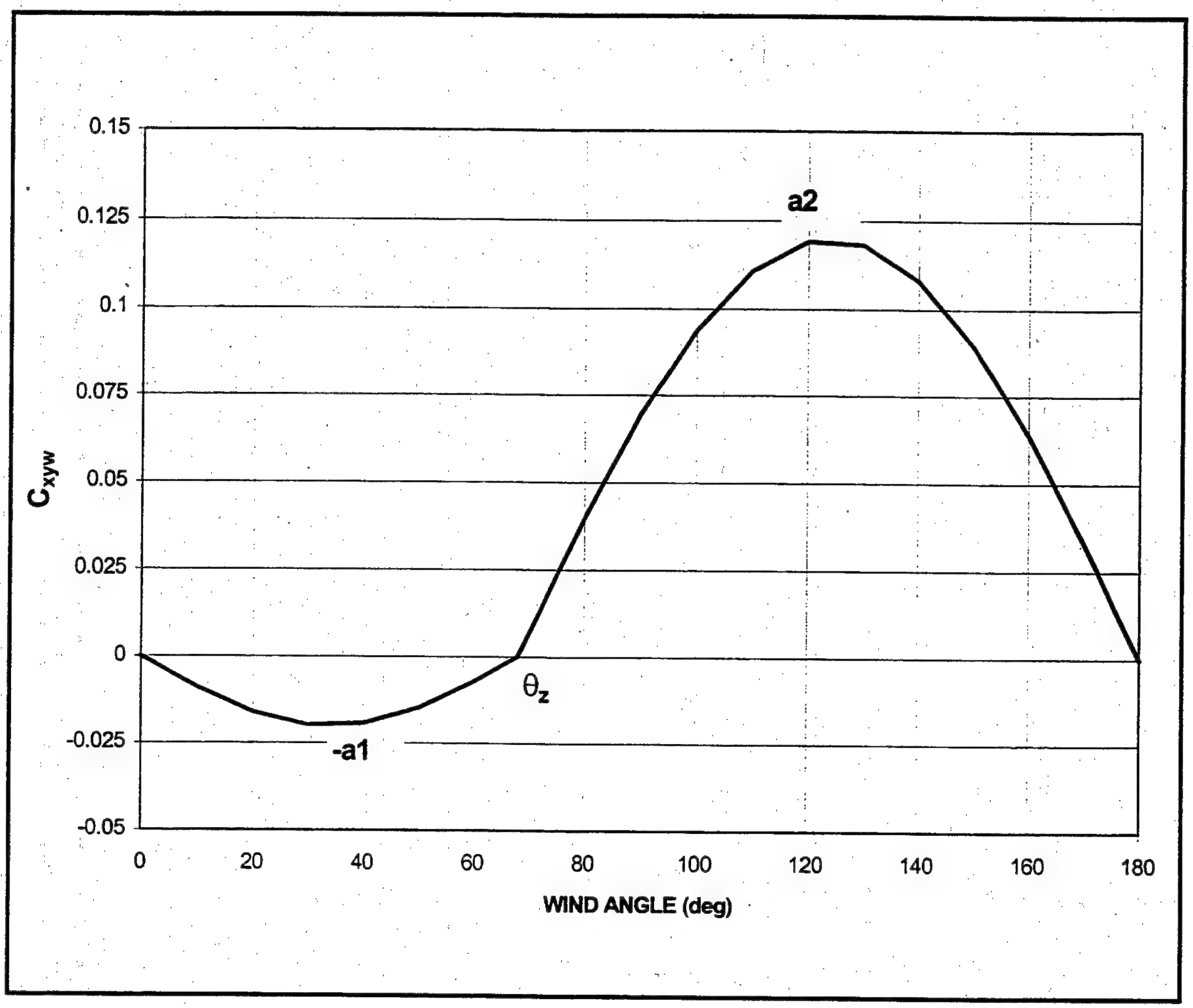



Figure 32
Sample Yaw Normalized Moment Coefficient

4.5 Static Current Forces/Moments. Methods to determine static current forces and moments on stationary moored vessels in the surge and sway directions and yaw moment are presented in this section. These planar directions are of primary importance in many mooring designs.

4.5.1 Static Transverse Current Force. The transverse current force is defined as that component of force perpendicular to the vessel centerline. If a ship has a large underkeel clearance, then water can freely flow under the keel, as shown in Figure 33(a). If the underkeel clearance is small, as shown in Figure 33(b), then the ship more effectively blocks current flow, and the transverse current force on the ship increases. These effects are considered and the transverse current force is determined from the equation:

EQUATION: $$F_{yc} = 0.5\,\rho_w\,V_c^2\,L_{wL}\,T\,C_{yc}\,\sin\theta_c \qquad (10)$$

where

$F_{yc}$ = transverse current force (newtons)
$\rho_w$ = mass density of water (from Table 20)
$V_c$ = current velocity (m/s)
$L_{wL}$ = vessel waterline length (m)
$T$ = average vessel draft (m)
$C_{yc}$ = transverse current force drag coefficient
$\theta_c$ = current angle (degrees)

The transverse current force drag coefficient as formulated in Broadside Current Forces on Moored Ships, Seelig et al. (1992) is shown in Figure 34. This drag coefficient can be determined from:

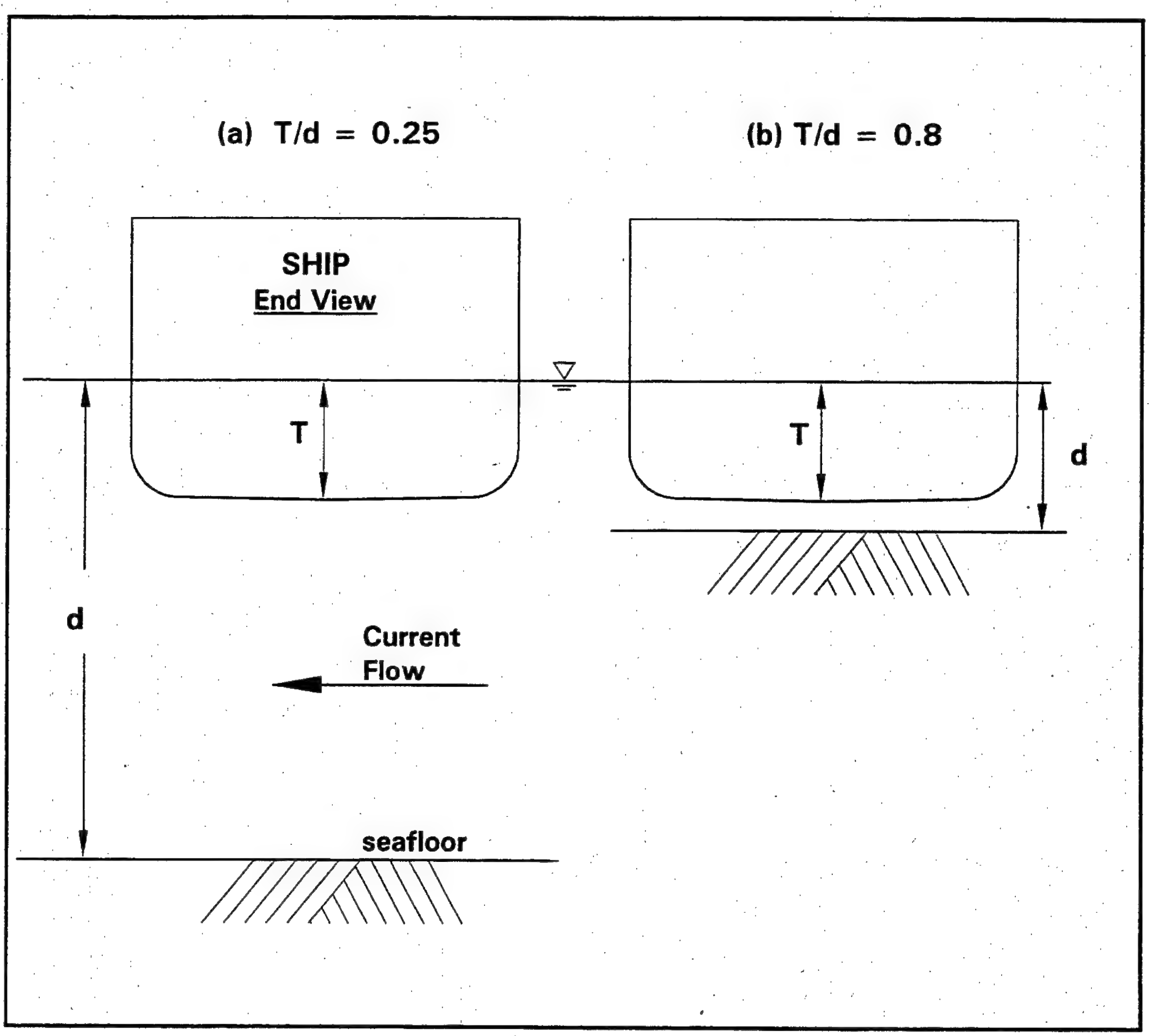



Figure 33
Examples of Ratios of Ship Draft (T) to Water Depth (d)

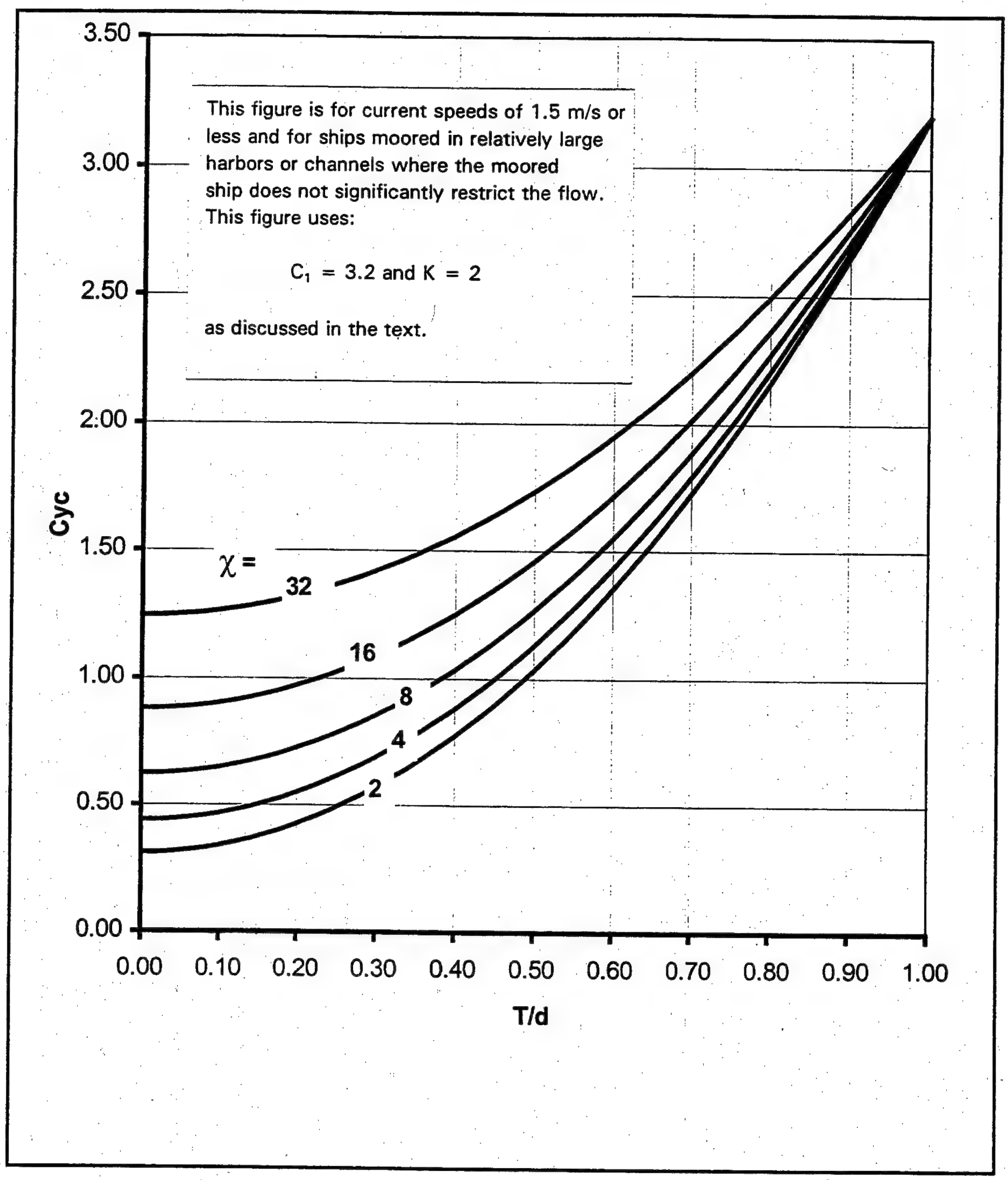



Figure 34
Broadside Current Drag Coefficient

EQUATION: $$C_{yc} = C_0 + (C_1 - C_0) * (T/d)^K \qquad (11)$$

where $C_0$ = deepwater current force drag coefficient for $T/d \approx 0.0$; this deepwater drag coefficient is estimated from:

EQUATION: $$C_0 = 0.22 * \sqrt{\chi} \qquad (12)$$

where $\chi$ is a dimensionless ship parameter calculated as:

EQUATION: $$\chi = L_{wL}^{\,2} * A_m / (B * V) \qquad (13)$$

where

$L_{wL}$ is the vessel length at waterline(m)

$A_m$ is the immersed cross-sectional area of the ship at midsection ($m^2$)

$B$ is the beam (maximum ship width at the waterline) (m), and

$V$ is the submerged volume of the ship (which can be found by taking the displacement of the vessel divided by the unit weight of water, given in Table 20 ($m^3$)).

$C_1$ = shallow water current force drag coefficient where $T/d = 1.0$; for currents of 1.5 m/s (3 knots or 5 ft/sec) or less

T = average vessel draft (m)

d = water depth (m)

K = dimensionless exponent; laboratory data from ship models shows:

K = 2 Wide range of ship and barge tests; most all of the physical model data available can be fit with this coefficient

K = 3 From a small number of tests on a fixed cargo ship and for a small number of tests on an old aircraft carrier, CVE-55

K = 5 From a small number of tests on an old submarine hull, SS-212

The immersed cross-sectional area of the ship at midships, $A_m$, can be determined from:

EQUATION: $$A_m = C_m * B * T \tag{14}$$

Values of the midship coefficient, $C_m$, are provided in the NAVFAC Ship's Database for DOD ships.

The above methods for determining the transverse current force are recommended for normal design conditions with moderate current speeds of 1.5 m/s (3 knots or 5 ft/sec) or less and in relatively wide channels and harbors (see Seelig et al., 1992).

If the vessel is moored broadside in currents greater than 1.5 m/s (3 knots or 5 ft/sec), then scale model laboratory data show that there can be significant vessel heel/roll, which effectively increases the drag force on the vessel. In some model tests in shallow water and at high current speeds this effect was so pronounced that the model ship capsized. Mooring a vessel broadside in a high current should be avoided, if possible.

Scale physical model tests show that a vessel moored broadside in a restricted channel has increased current forces. This is because the vessel decreases the effective flow area of a restricted channel, which causes the current speed and current force to increase.

For specialized cases where:

(1) vessels are moored in current of 1.5 m/s (3 knots or 5 ft/sec) or more, and/or

(2) for vessels moored in restricted channels

then the designer should contact the Moorings Center of Expertise, NFESC ECDET, Washington Navy Yard Bldg. 218, 901 M St. SE, Washington DC 20374-5063.

EXAMPLE: Find the current force on an FFG-7 vessel produced by a current of $\theta_c$=90 degrees to the ship centerline with a speed of 1.5 m/s (2.9 knots or 4.9 ft/sec) in salt water for a given ship

draft. At the mooring location, the harbor has a cross-sectional area much larger than the submerged ship longitudinal area, $L_{wL}*T$.

SOLUTION: Dimensions and characteristics of this vessel are summarized in the lower right portion of Figure 35. Transverse current drag coefficients predicted using Equation 11 are shown on this figure as a solid bold line. Physical scale model data (U.S. Naval Academy (USNA), EW-9-90, Evaluation of Viscous Damping Models for Single Point Mooring Simulation) are shown as symbols in the drawing, showing that Equation 11 provides a reasonable estimate of drag coefficients. Predicted current forces for this example are given in Table 26.

Table 26
Predicted Transverse Current Forces on FFG-7
for a Current Speed of 1.5 m/s (2.9 knots)

| T/d | d (m) | D (ft) | Fyc (MN)* | Fyc (kips)** |
|---|---|---|---|---|
| 0.096 | 45.7 | 150 | 0.55 | 123 |
| 0.288 | 15.2 | 50 | 0.66 | 148 |
| 0.576 | 7.62 | 25 | 1.03 | 231 |
| 0.72 | 6.096 | 20 | 1.30 | 293 |
| 0.96 | 4.572 | 15 | 1.90 | 427 |

* MN = one million newtons
**kip = one thousand pounds force

This example shows that in shallow water the transverse current force can be three times or larger than in deep water for an FFG-7.

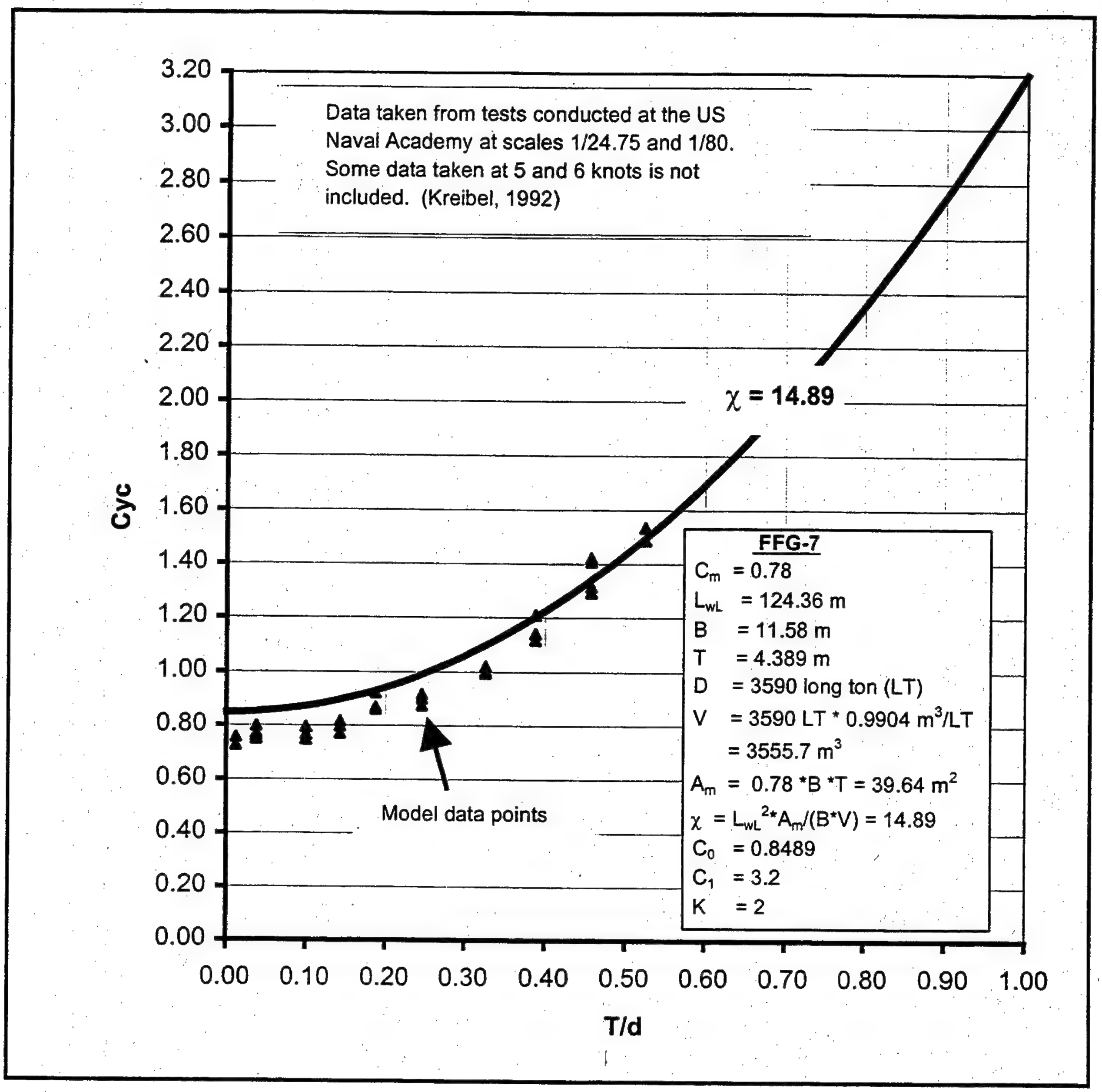



Figure 35
Example of Transverse Current Drag Coefficients

4.5.2 Static Longitudinal Current Force. The longitudinal current force is defined as that component of force parallel to the centerline of the vessel. This force is determined from the following equation (Naval Civil Engineering Laboratory (NCEL), TN-1634, STATMOOR - A Single-Point Mooring Static Analysis Program):

EQUATION: $$F_{xc} = F_{x\,FORM} + F_{x\,FRICTION} + F_{x\,PROP} \qquad (15)$$

where

$F_{xc}$ = total longitudinal current load (newtons)
$F_{xFORM}$ = longitudinal current load due to form drag (newtons)
$F_{xFRICTION}$ = longitudinal current load due to skin friction (newtons)
$F_{xPROP}$ = longitudinal current load due to propeller drag (newtons)

The three elements of the general longitudinal current load equation, $F_{xFORM}$, $F_{xFRICTION}$, and $F_{xPROP}$ are described below:

$F_{xFORM}$ = longitudinal current load due to form drag

EQUATION: $$F_{xFORM} = \frac{1}{2}\rho_w V_c^2 B\, T\, C_{xcb} \cos(\theta_c) \qquad (16)$$

where

$\rho_w$ = mass density of water, from Table 20
$V_c$ = current speed (m/s)
B = maximum vessel width at the waterline(m)
T = average vessel draft (m)
$C_{xcb}$ = longitudinal current form drag coefficient = 0.1
$\theta_c$ = current angle (degrees)

$F_{xFRICTION}$ = longitudinal current load due to skin friction

EQUATION: $$F_{xFRICTION} = \frac{1}{2}\rho_w V_c^2 S C_{xca} \cos(\theta_c) \qquad (17)$$

where

$\rho_w$ = mass density of water, from Table 20
$V_c$ = current speed (m/s)
S = wetted surface area ($m^2$); estimated using

$$S = 1.7\, T\, L_{wL} + \left(\frac{D}{T\gamma_W}\right) \qquad (18)$$

T = average vessel draft (m)
$L_{wL}$ = waterline length of vessel (m)
D = ship displacement (newtons)
$\gamma_W$ = weight density of water, from Table 21
$C_{xca}$ = longitudinal skin friction coefficient, estimated using:

$$C_{xca} = 0.075 / \left(\left(\log_{10} R_N\right) - 2\right)^2 \qquad (19)$$

$R_N$ = Reynolds Number

$$R_N = \left|\frac{V_c L_{wL} \cos(\theta_c)}{\nu}\right| \qquad (20)$$

$\nu$ = kinematic viscosity of water, from Table 21
$\theta_c$ = current angle (degrees)

$F_{xPROP}$ = longitudinal current load due to fixed propeller drag

EQUATION: $$F_{xPROP} = \frac{1}{2}\rho_w V_c^2 A_p C_{PROP} \cos(\theta_C) \qquad (21)$$

where

$\rho_w$ = mass density of water, from Table 21
$V_c$ = current speed (m/s)
$A_p$ = propeller expanded blade area ($m^2$)
$C_{PROP}$ = propeller drag coefficient = 1.0
$\theta_C$ = current angle (degrees)

$$A_p = \frac{A_{Tpp}}{1.067 - 0.229\,(p/d)} = \frac{A_{Tpp}}{0.838} \qquad (22)$$

$A_{Tpp}$ = total projected propeller area ($m^2$) for an assumed propeller pitch ratio of $p/d = 1.0$

$$A_{Tpp} = \frac{L_{wL}\,B}{A_R} \qquad (23)$$

$A_R$ is a dimensionless area ratio for propellers. Typical values of this parameter for major vessel groups are given in Table 27.

Table 27
$A_R$ for Major Vessel Groups

| SHIP | AREA RATIO, $A_R$ |
|---|---|
| Destroyer | 100 |
| Cruiser | 160 |
| Carrier | 125 |
| Cargo | 240 |
| Tanker | 270 |
| Submarine | 125 |

Note that in these and all other engineering calculations discussed in this handbook, the user must be careful to keep units consistent.

EXAMPLE: Find the longitudinal current force with a bow-on current of $\theta_c$=180 degrees with a current speed of 1.544 m/sec (3 knots) on a destroyer in salt water with the characteristics shown in Table 28.

SOLUTION: Table 29 shows the predicted current forces. Note that these forces are negative, since the bow-on current is in a negative "X" direction. For this destroyer, the force on the propeller is approximately two-thirds of the total longitudinal current force. For commercial ships, with relatively smaller propellers, form and friction drag produce a larger percentage of the current force.

Table 28
Example Destroyer

| PARAMETER | SI SYSTEM | ENGLISH OR INCH-POUND SYSTEM |
|---|---|---|
| $L_{wL}$ | 161.2 m | 529 ft |
| T | 6.4 m | 21 ft |
| B | 16.76 m | 55 ft |
| D, ship displacement | 7.93E6 kg | 7810 long tons |
| $C_m$; estimated | 0.83 | 0.83 |
| S; est. from Eq 18 | 2963 $m^2$ | 31897 $ft^2$ |
| $A_R$; from Table 27 | 100 | 100 |
| $R_N$; from Eq 20 | 2.09E8 | 2.09E8 |
| $C_{xca}$; est. from Eq 19 | 0.00188 | 0.00188 |
| Ap; est. from Eq 22 | 32.256 $m^2$ | 347.2 $ft^2$ |

Table 29
Example Longitudinal Current Forces on a Destroyer

| FORCE | SI SYSTEM | ENGLISH OR INCH-POUND SYSTEM | PERCENT OF TOTAL FORCE |
|---|---|---|---|
| $F_{xFORM}$; Eq 15 | -13.1 kN* | -2.95 kip** | 22% |
| $F_{xFRICTION}$; Eq 16 | -6.8 kN | -1.53 kip | 12% |
| $F_{xPROP}$; Eq 17 | -39.4 kN | -8.87 kip | 66% |
| Total $F_{xc}$ = | -59.4 kN | -13.4 kip | 100% |

* kN = one thousand newtons
**kip = one thousand pounds force

4.5.3 Static Current Yaw Moment. The current yaw moment is defined as that component of moment acting about the vessel's vertical "Z"-axis. This moment is determined from the equation:

EQUATION:
$$M_{xyc} = F_{yc}\left(\frac{e_c}{L_{wL}}\right)L_{wL} \tag{24}$$

where

$M_{xyc}$ = current yaw moment (newton*m)
$F_{yc}$ = transverse current force (newton)
$\frac{e_c}{L_{wL}}$ = ratio of eccentricity to vessel waterline length
$e_c$ = eccentricity of $F_{yc}$ (m)
$L_{wL}$ = vessel waterline length (m)

The dimensionless moment arm $\frac{e_c}{L_{wL}}$ is calculated by choosing the slope and y-intercept variables from Table 30 which are a function of the vessel hull. The dimensionless moment arm is dependent upon the current angle to the vessel, as shown in Equation 25:

EQUATION:
$$\frac{e}{L_{wL}} = a + b * \theta_c \qquad \theta_c = 0° \text{ to } 180° \tag{25}$$

$$\frac{e}{L_{wL}} = -a - (b * (360\deg - \theta_c)) \qquad \theta_c = 180° \text{ to } 360° \tag{25a}$$

where

$\frac{e_c}{L_{wL}}$ = ratio of eccentricity to vessel waterline length
$a$ = y-intercept (refer to Table 30) (dimensionless)
$b$ = slope per degree (refer to Table 29)
$\theta_c$ = current angle (degrees)

The above methods for determining the eccentricity ratio are recommended for normal design conditions with moderate current speeds of less than 1.5 m/s (3 knots or 5 ft/sec). Values provided in Table 30 are based upon least squares fit of scale model data taken for the case of ships with level keels. Data are not adequately available for evaluating the effect of trim on the current moment.

Table 30
Current Moment Eccentricity Ratio Variables

| SHIP | a<br>Y-INTERCEPT | b<br>SLOPE PER DEGREE | NOTES |
|---|---|---|---|
| SERIES 60 | -0.291 | 0.00353 | Full hull form typical of cargo ships |
| FFG | -0.201 | 0.00221 | "Rounded" hull typical of surface warships |
| CVE-55 | -0.168 | 0.00189 | Old attack aircraft carrier |
| SS-212 | -0.244 | 0.00255 | Old submarine |

4.6 Wind and Current Forces and Moments on Multiple Ships. If ships are moored in close proximity to one another then the nearby ship(s) can influence the forces/moments on a given ship. The best information available on the effects of nearby ships are results from physical model tests, because the physical processes involved are highly complex. Appendix A provides scale model test results of wind and current forces and moments for multiple identical ships. From two to six identical ships were tested and the test results were compared with test results from a single ship. Data are provided for aircraft carriers, destroyers, cargo ships, and submarines.

Cases included in Appendix A include: individual ships, ships in nests and ships moored on either sides of piers. Results are provided for the effects of winds and currents in both tabular and graphical form.

# Appendix C

# Underwater Inspection Criteria

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# UNDERWATER INSPECTION CRITERIA


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## EXECUTIVE SUMMARY

The Marine Facilities Division (MFD) of the California State Lands Commission (CSLC) is in the process of reviewing and formulating various design and inspection criteria for waterfront facilities. The MFD has a regulatory requirement to require a thorough examination of each marine terminal in the State to determine whether the structural integrity of the terminal, the oil transfer operations system, and the safety equipment are designed and being maintained in a safe working condition. To meet this regulatory objective the CSLC has developed a procedure for performing an in-depth structural and safety system audit of existing marine loading and discharge terminals located onshore, near-shore, and offshore California. These procedures apply to pier and wharf terminals, and offshore multi-point mooring marine terminals.

The Naval Facilities Engineering Service Center (NFESC) has been tasked to provide input to the review and formulation of design and inspection criteria for waterfront facilities, based on the Navy's extensive experience and expertise in this area. This document addresses the underwater inspection component of the overall effort. This underwater inspection criteria is intended to provide guidance to the CSLC on the inspection of the underwater components of a marine oil terminal facility with the intent on identifying structural damage or weaknesses that might affect the continued fitness-for-purpose of the terminal.

TABLE OF CONTENTS

## TABLE OF CONTENTS (continued)

Page

**TABLE OF CONTENTS** (continued)

TABLE OF CONTENTS (continued)

## 1.0 INTRODUCTION

This California State Lands Commission's (CSLC) has the following regulatory requirement:

> "At least once every three years, the Marine Facilities Division (MFD) shall cause to be carried out a thorough examination of each marine terminal in the state to determine whether the structural integrity of the terminal, the oil transfer operations system and the safety equipment are designed and being maintained in a safe working condition."

To meet this regulatory objective the CSLC has developed a procedure for performing an in-depth structural and safety system audit of existing marine loading and discharge terminals located onshore, near-shore and offshore California. These procedures apply to pier and wharf terminals, and offshore multi-point mooring marine terminals. Mooring equipment on the vessel used to secure the vessel to the wharf/pier or offshore multi-point mooring is a part of this audit procedure, too.

The objective of the audit should be prevention as well as cure. In addition to the correction of an individual non-conformance item, the audit team should look for improvements to the safety system or structure, which would prevent its recurrence elsewhere. Ideally, the participants in the audit work as a team and the objective of the audit is not only to document and assess the criticality of deficiencies, but also to enhance reliability, safety and structural integrity of the terminal and its operation.

Terminal audits should compare the facility with the standards and practices used for its original design and operation. However, it should also be compared against current standards and those areas where upgrading would provide a significant improvement in safety should be identified. The purpose of the audit procedure is to:

(1) Identify safety system, mechanical, and electrical deficiencies at the marine terminal,

(2) Identify structural damage or weaknesses that might affect the continued fitness-for-purpose of the terminal,

(3) Advise whether these deficiencies have been properly assessed, and,

(4) Advise what steps should be taken to prevent, or minimize these potential risks.

This underwater inspection criteria is intended to provide guidance to the CSLC on the inspection of the underwater components of a marine oil terminal facility with the intent on identifying structural damage or weaknesses that might affect the continued fitness-for-purpose of the terminal.

## 2.0 GENERAL CONSIDERATIONS

The fundamental purpose of any inspection is to provide the information necessary to assess the condition (capacity, safety, and rate of deterioration) of a structure. A waterfront inspection encompasses the examination of structures such as: piers, pilings, wharves, quaywalls, fender systems, dolphins, dry docks, and coastal protection structures. The usefulness of an inspection depends upon establishing a clear and complete record. Although the level of inspection will determine the extent of information to be provided, in general the inspection will address the following:

(a) Identification and description of all major damage and deterioration of the facility.

(b) Estimate of the extent of damage and deterioration.

(c) Identification of any problems associated with mobilization of equipment, personnel, and materials to accomplish repairs/maintenance.

(d) Updated layouts of pile plans (which occasionally differ significantly from the drawings available at the activity).

(e) Documentation of types and extent of marine growth (to help plan future inspections), as well as damage caused by their presence.

(f) Water depths at each facility.

(g) Water visibility, tidal range, and water current.

(h) Information for the database of waterfront facilities and data to assist in planning future inspections.

(i) Assessment of general physical condition including projected load capacities of the in-water structures of each facility inspected.

(j) Recommendations for required maintenance and repair (M&R).

(k) Budgetary estimates of costs of this M&R, including examples of the derivation of the estimates.

(l) Estimate of expected life of each facility.

(m) Recommendations for types and frequencies of future underwater inspections.

There are several types of inspections, including:

(a) Baseline - to obtain data on an uninspected facility. This type involves the greatest "pre-inspection" effort.

(b) Routine - to obtain data on general condition, confirm drawings, estimate repair costs, etc.

(c) Design Survey - to obtain data for specifications or for detailed cost estimates.

(d) Acceptance - to obtain data confirming that a repair has been completed according to plan or specification.

(e) Research - to obtain data for research on deterioration rates, etc.

A number of Government reference documents dealing with waterfront inspections exist. The inspection procedures and planning factors outlined in this document have been taken from several of them. These references are listed in the bibliography at the end of this document.

## 3.0 SCOPE

Underwater inspections are primarily visual observations of the facility being inspected. Quantitative measurements, such as underwater voltmeter readings on metal structures and thickness measurements on mooring chain and steel piling, are often taken. Before making the observation, it is usually necessary to clean the structure of marine growth and fouling. Several techniques are used to accomplish this cleaning, ranging from hand cleaning with scrapers and wire brushes to the use of waterblasting jets and hydraulically powered mechanical abrasive tools.

This document has been arranged to present a general description of:

- Waterfront facilities (Chapter 4)
- General inspection procedures (Chapter 5)

And detailed descriptions of the procedures to be used for:

- Inspecting steel structures (Chapter 6)
- Compliant moorings (Chapter 7)
- Concrete structures (Chapter 8)
- Timber structures (Chapter 9)
- Stone masonry structures (Chapter 10)
- Coastal protection structures (Chapter 11)
- Synthetic Materials and Components (Chapter 12)
- Quaywalls (Chapter 13)

Each of these sections also includes a description of the causes of deterioration of the relevant type of structural material.

## 4.0 INTRODUCTION TO WATERFRONT FACILITIES

The following discussion provides a very brief introduction to the types of waterfront facilities that may be encountered. The following Government handbooks provide useful information:

MIL-HDBK-1025/1 - Piers and Wharves
MIL-HDBK-1025/6 - General Criteria for Waterfront Construction
MO-104.1 - Maintenance of Fender Systems and Camels
MO-104.2 - Specialized Underwater Waterfront Facilities Inspections
MO-306 - Maintenance and Operation of Cathodic Protection Systems
DM-26.1 - Harbors
DM-26.5 - Fleet Moorings

Marine facilities include:

- Berthing facilities
- Drydocks
- Coastal protection structures
- Components of waterfront structures: fender systems, piling, and dolphins
- Compliant moorings
- Underwater cables and pipelines

Berthing facilities and coastal protection structures are described in more detail below.

### 4.1 Berthing Facilities

Berthing facilities provide space for: mooring, shore utilities, hotel services, loading and unloading of cargo, personnel, ordnance, and fuel, and maintenance, repair, and fitting out. Piers are also used to support specific functions such as magnetic silencing facilities for submarines. Some typical configurations of piers and wharves are shown in Figure 4-1.

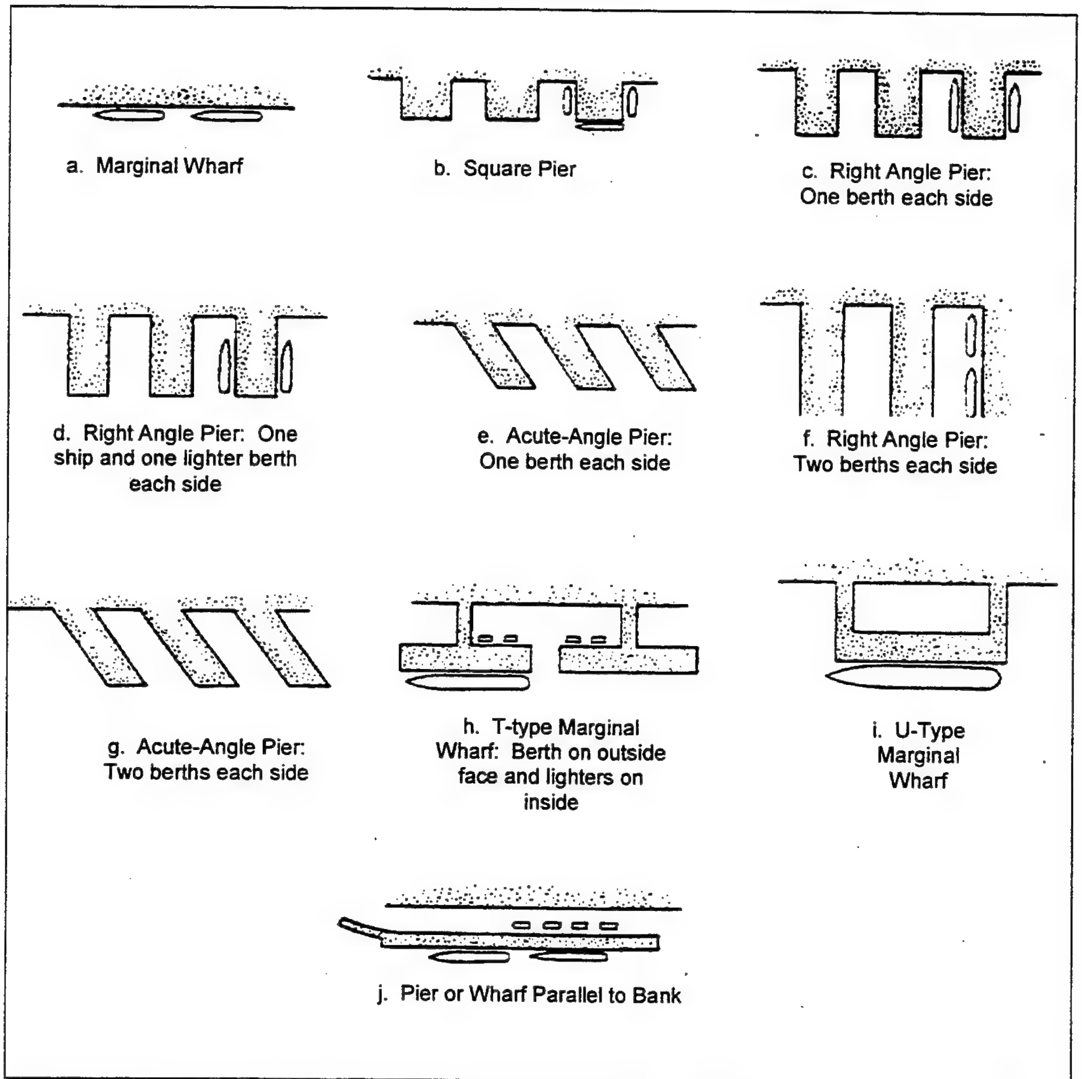


Figure 4-1. Typical configurations of piers and wharves.

**4.1.1 Piers.** Piers are docks that extend outward from the shore into the water. There are basically four types of pier structures with distinct differences in configuration: open, closed, combination, and floating. These piers are:

(a) Open piers are pile-supported platform structures which allow water to flow underneath. Conventionally open piers are single deck structures although some are double deck.

(b) Closed piers are constructed so that water is prevented from flowing underneath. The solid fill pier is surrounded along the perimeter by a bulkhead that holds back the fill.

(c) Floating piers can be constructed of steel or concrete and are connected to the shore with access ramps. Guide piles or anchor systems prevent lateral movement. Floating piers may be either single or double deck.

**4.1.2 Piling.** Piling is a common element found on piers, wharves, and some fender systems. Figure 4-2 provides some typical pile cross sections for steel, wood, and concrete piles with dimensions typically found in marine structures.

The basic types of piling are:

(a) Vertical bearing piles are used to support the dead weight of the pier as well as the live loads on the pier.

(b) Batter piles primarily provide lateral and longitudinal stability but do provide limited load carrying capacity.

(c) Fender piles absorb the impact of berthing ships.

(d) Sheet piling is used with various waterfront facilities to retain fill.

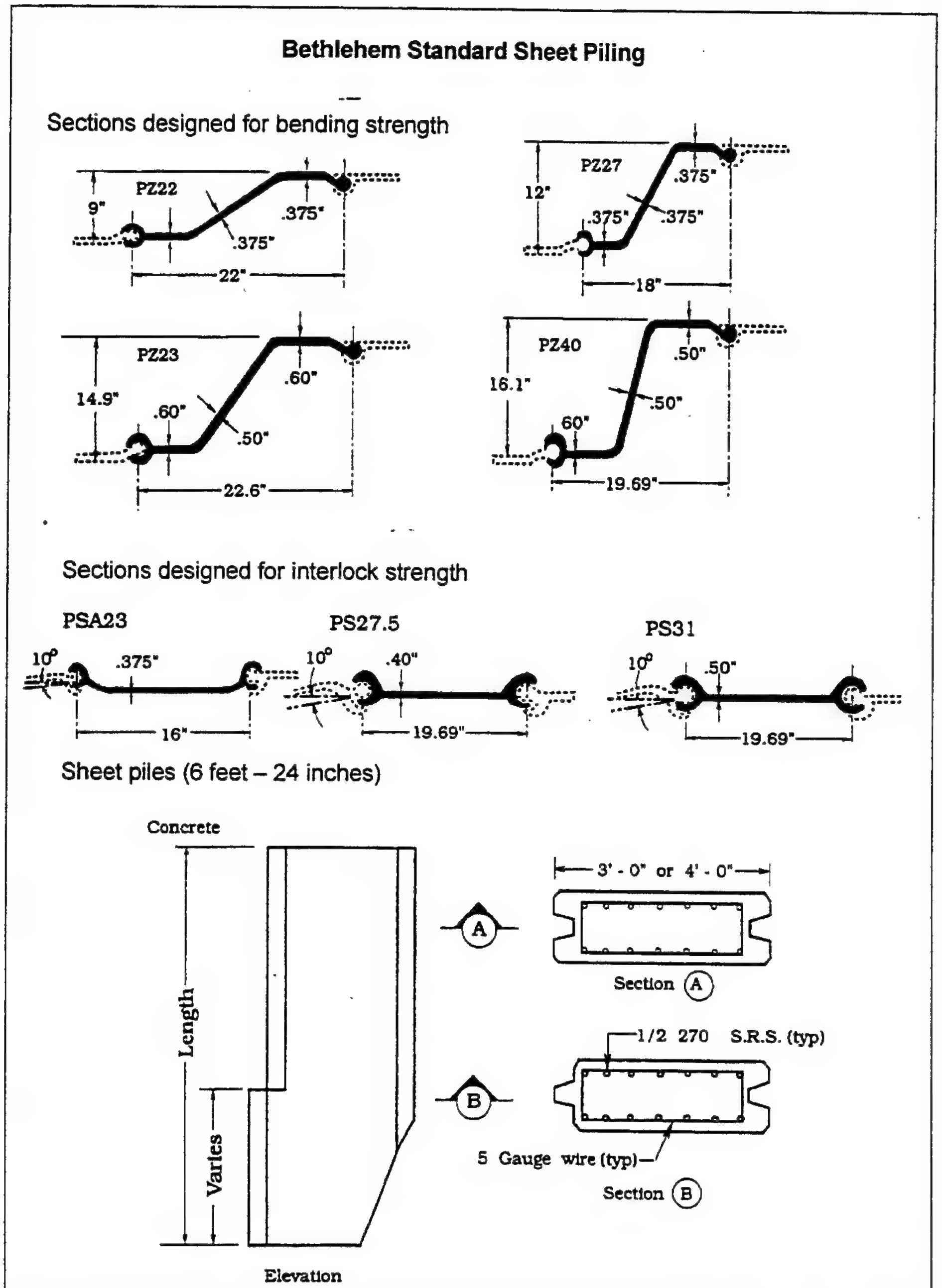


Figure 4-2. Typical pile cross sections for steel, wood, and concrete piles.

**4.1.3 Wharves and Quaywalls.** Wharves are docks which are oriented approximately parallel to the shore and are connected to shore along their entire length. The retaining structure used to contain the backfill is commonly referred to as the quaywall or bulkhead. Several types of these structures are shown in Figure 4-3.

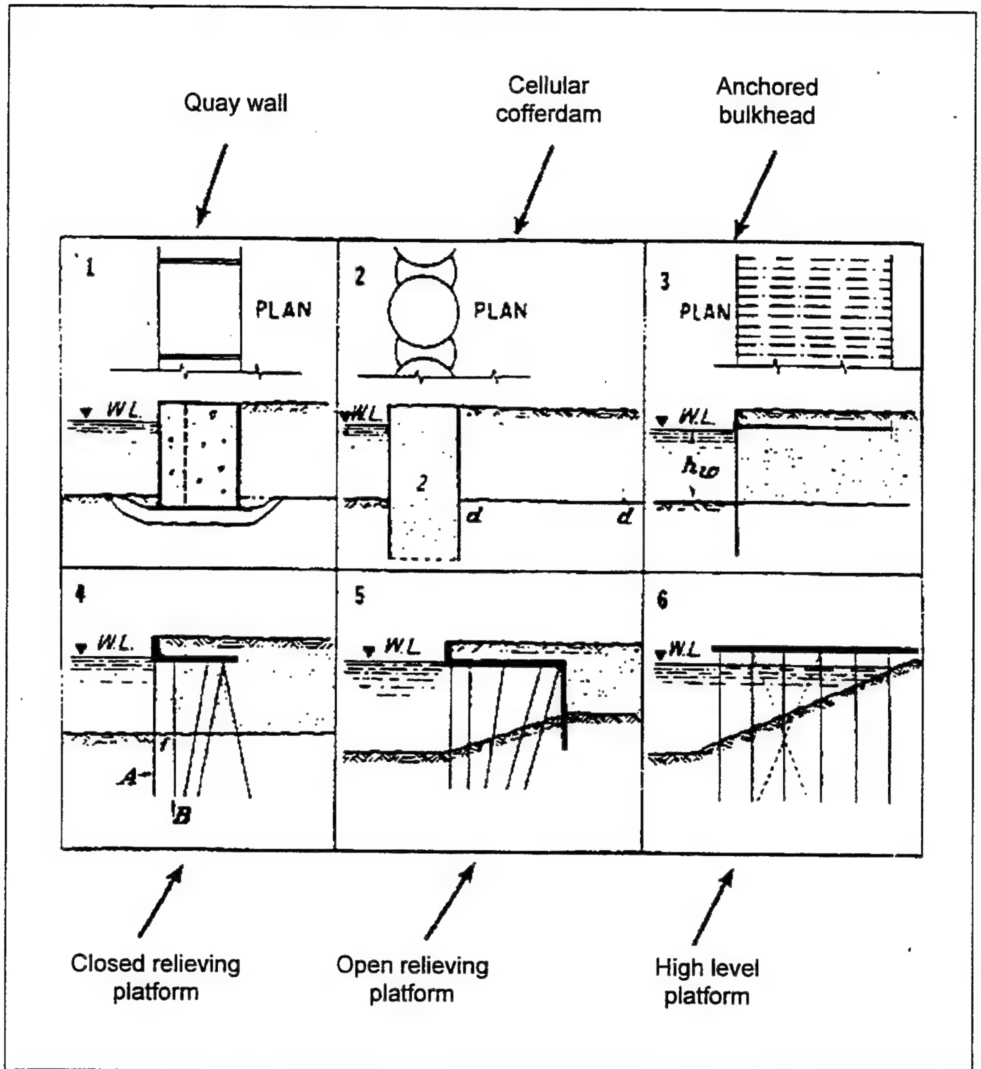


Figure 4-3. Types of Quaywalls/Bulkheads.

**4.1.4 Fender Systems.** Fender systems are used on piers to protect the ship and the pier during berthing operations and while the ship is moored. On relatively inflexible piers and wharves the fender acts as a buffer in absorbing or dissipating the impact energy of the ship without damaging the ship. Where ships are berthed against pile-supported structures, protection of the structure is of more serious concern. The main type of fenders and components that are found on older and smaller piers are fender pile systems. For modern larger piers, various types of fenders are attached to the pier, and they include:

- Rubber units in compression or shear (various shapes: cylindrical, rectangular, trapezoidal, wing, etc.).

- Buckling column (various shapes).

- Pneumatic (air filled) shapes.

- Foam filled (typically cylindrical shape).

**4.1.5 Dolphins.** Dolphins (Figure 4-4) are groups of piles placed near piers and wharves or in turning basins and ship channels. These structures are used to guide vessels into their moorings, to mark underwater structures, to moor vessels to, to berth vessels against, and to support navigational aids.

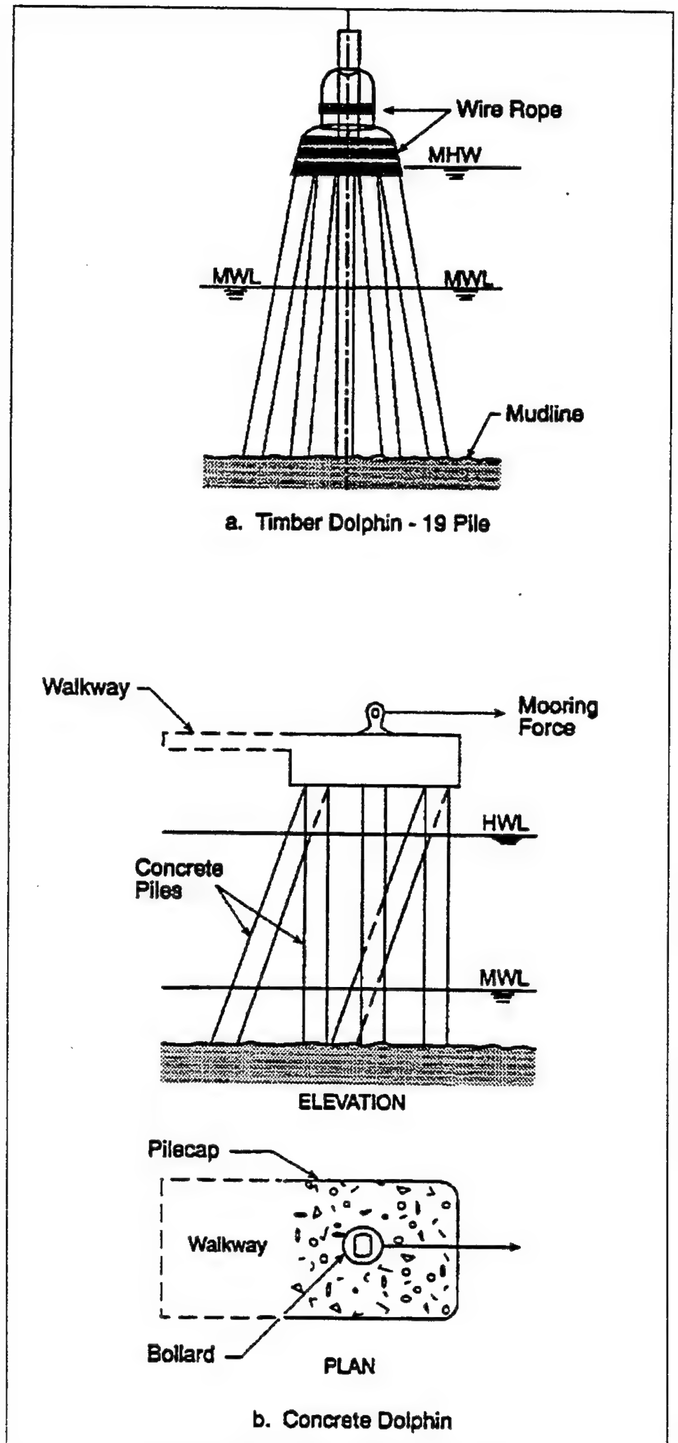


Figure 4-4. Mooring Dolphins.

### 4.2 Coastal Protection Structures

Coastal protection structures are designed to reduce the effects of wave action, so as to protect harbors and reduce the formation of sandbars. They can be fabricated out of a variety of materials including concrete, rock rubble, granite masonry, and reinforced precast concrete armor units as shown in Figure 4-5. Typical coastal structures include seawalls, groins, jetties, and breakwaters. These structures are:

(a) Seawalls are massive coastal structures built along the shoreline. Their primary function is to protect areas from erosion caused by waves or flooding.

(b) Groins (Figure 4-6) are designed to minimize coastal erosion by controlling the rate of shifting sand by influencing offshore currents and waves. Groins project outward, perpendicular to the shoreline.

(c) Jetties extend outward from shore to prevent the formation of sandbars and direct the flow of water from currents, tides, and waves.

(d) Breakwaters are generally located outside the entrance of a harbor, anchorage, or coastline. They are designed primarily to protect the inner waters and shoreline from the effects of heavy seas. Breakwaters may be connected or detached from the shore.

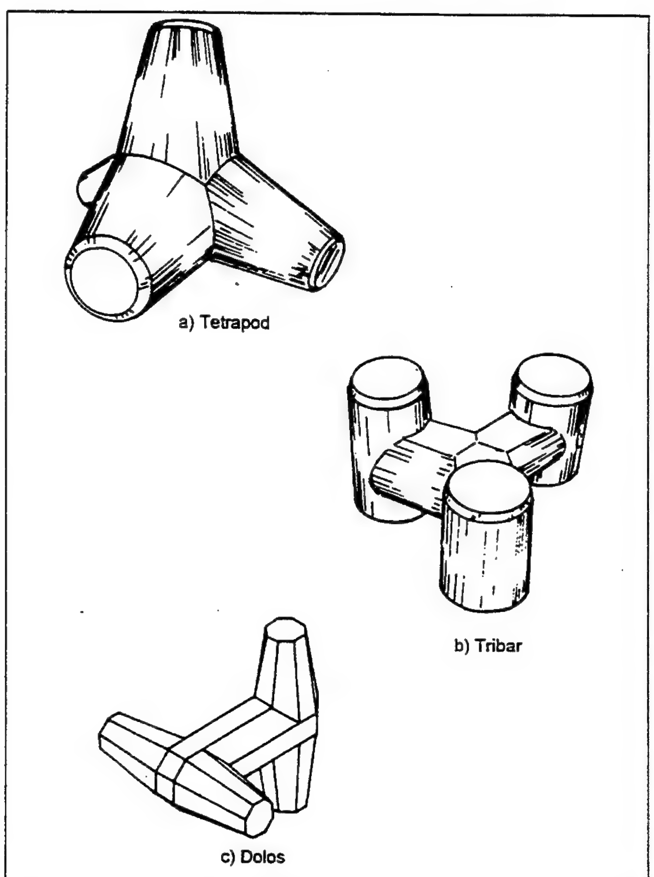


Figure 4-5. Precast Concrete Armor Units used in Jetties, Breakwaters, and Groins.

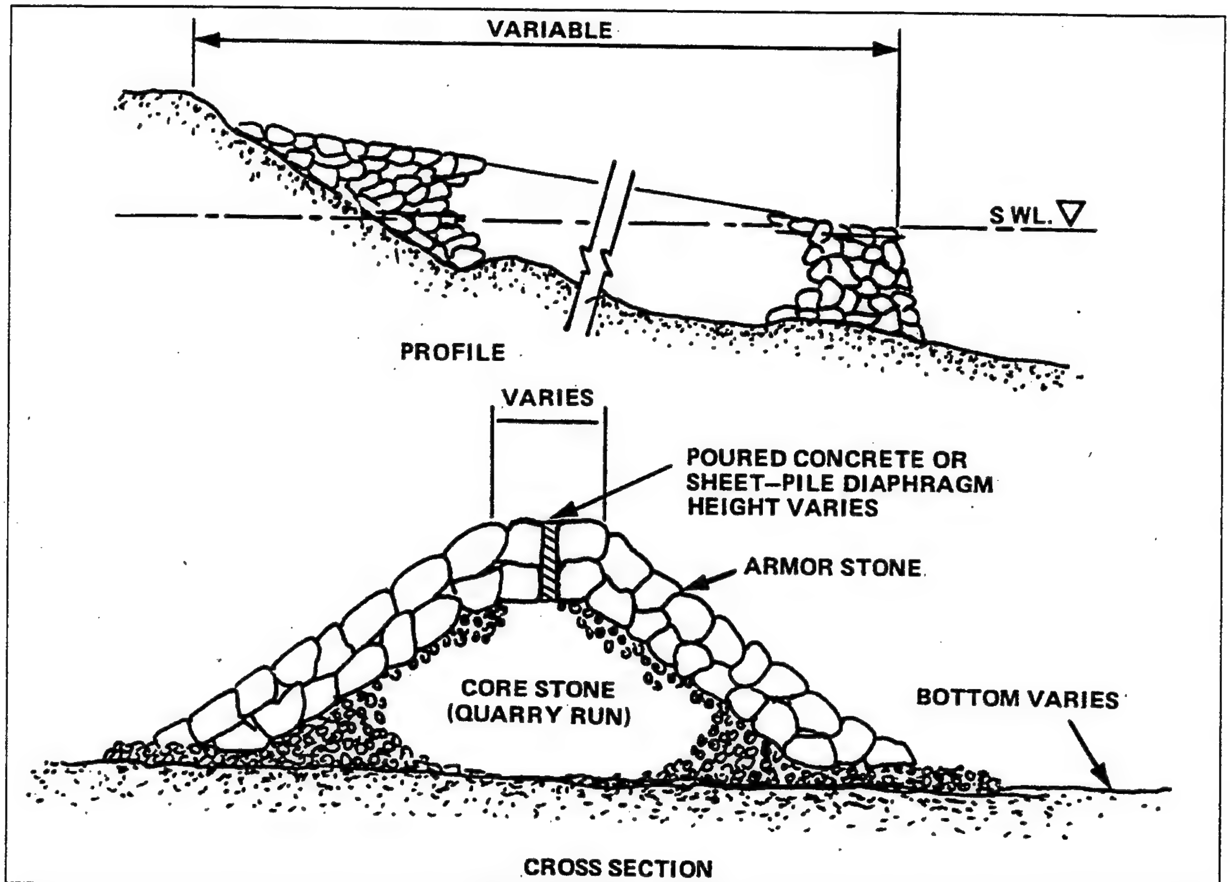


Figure 4-6. Groins.

## 5.0 GENERAL INSPECTION PROCEDURES

The purpose of any inspection is to provide the information necessary to assess the condition (capacity, safety, and rate of deterioration) of a structure. The usefulness of an inspection depends, therefore, on the suitability and recording of the data (observations) obtained for use in later engineering evaluations. An underwater inspection is a condition survey; therefore, the diver should make and report observations and measurements that can be used by an engineer to make the engineering assessments. Ideally, the engineer making the engineering assessments is also a diver and dives on the facility for at least a portion of the inspection. A part of the engineering assessment will be to determine the cause of the failure or damage; therefore, detailing and documenting the inspection is important.

For example, the diver should observe, measure, and report that a sheet pile wall has a hole measuring 2 feet by 3 feet, at a depth of 2 feet below mean low water (MLW), and that behind the hole is a 6-foot deep cavity. These data may be supplemented by the diver's opinion regarding the structural adequacy of the wall if the nature of its condition is obvious. A final conclusion, however, can only be reached after an analysis of the structure's existing condition has been conducted.

Since divers' observations and measurements are often the only data available for the topside personnel to make an engineering assessment, the reliability of such data is critically important. Therefore, the quality control of the measurements becomes an important issue. At least 10 percent of all measurements and observations must be rechecked by a second diver to

ensure accuracy. If any discrepancy is discovered, all measurements and observations must be rechecked.

An important part of any inspection operation is the recording of the diver's observations. Observations, both qualitative and quantitative, can be recorded underwater on a Plexiglas slate with a grease pencil. However, direct hardwire communication between the diver and topside is much more efficient. In addition, use a video recording system, a photographic camera, and a voice recorder whenever possible. The dive supervisor or inspection team leader should debrief the working diver as soon after the dive as possible. This valuable information should be recorded for later reference.

### 5.1 Levels of Inspection

Three basic types or levels of inspection are used for inspecting marine facilities. The resources and preparation needed to do the work distinguish the level of inspection. Also, the level of inspection determines the type of damage/defect that is detectable:

- Level I - General Visual Inspection. The Level I effort can confirm as-built structural plans and detect obvious major damage or deterioration due to overstress (collisions, ice), severe corrosion, or extensive biological growth and attack. The Level I will provide initial input for an inspection strategy. Although this is an overview, close attention should be given to confirming or providing information to update available facility drawings and condition evaluations.

This type of inspection does not involve cleaning of any structural elements and can therefore be conducted much more rapidly than the other types of inspections. The Level I effort is essentially a general inspection "swim-by" overview. It does not involve cleaning of structural elements, which allows the inspection to be conducted rapidly. The underwater inspector relies primarily on visual and/or tactile observations (depending on water clarity) to make condition assessments. These observations are made over the specified exterior surface area of the underwater structure, whether it is a quaywall, bulkhead, seawall, pile, or mooring.

- Level II - Close-Up Visual Inspection. Level II efforts are complete, detailed investigations of selected components or subcomponents, or critical areas of the structure, directed toward detecting and describing damaged or deteriorated areas that may be hidden by surface biofouling. Limited deterioration measurements are obtained. These data are sufficient for gross estimates of facility load capability. This type of inspection will generally involve prior or concurrent cleaning of part of the structural elements. Since cleaning is time consuming, it is generally restricted to areas that are critical or that may be typical of the entire structure. The amount and thoroughness of cleaning to be performed are governed by what is necessary to determine the general condition of the overall facility. Simple instruments such as calipers and measuring scales are commonly used to take physical measurements. Subjective judgments of structural integrity are occasionally made by probing wood with ice picks and by pounding concrete with hammers.

- Level III - Highly Detailed Inspection. This level of inspection is primarily designed to provide data that can be used to perform a structural assessment and will often require the use of Nondestructive Testing (NDT) techniques. The procedures are conducted to detect hidden or imminent damage, loss in cross-sectional area, and material homogeneity. The training, cleaning, and testing requirements will vary depending on the type of damage/defect that is to be investigated and the type of inspection equipment to be used. A Level III examination will

normally require prior cleaning. In some cases, Level III inspections will require the use of partially destructive techniques such as sample coring in wood or concrete, material sampling, and in-situ surface hardness. The use of Level III inspection techniques is usually limited to key structural areas that may be suspect, or to structural areas that may be representative of the overall structure. Level III inspections will require considerably more experience and training than Level I or Level II inspections, and should be accomplished by qualified engineering or testing personnel. This type of inspection is covered in MO-104.2 (see Bibliography).

On steel H-piles, pipe piles, and sheet piles, metal thickness measurements are made with ultrasonic thickness equipment. In addition, electrical potential measurements, using a half-cell, are taken to verify the performance of the cathode protection system for steel structures. Concrete surfaces can be evaluated for hardness using a rebound hammer. A magnetic rebar locator can be used to establish the location and depth of rebar. There are few underwater instruments currently available for assessment of the interior of wood structures. Wood is inspected using calipers, ice picks, and hammers, and in some cases an incremental borer is used to obtain a core sample.

Table 5-1 summarizes the type of damage that is detectable with the three types of inspection. The level of inspection to be used for a particular task is usually decided early in the planning phase. However, depending upon visibility, marine growth, and extent of deterioration, this may be adjusted as the inspection proceeds. Often, the requirements of the local staff civil engineer or other authority will dictate the level of inspection. The underwater inspection may be accomplished by a qualified engineering diver or by a qualified, certified diver, supervised by an engineer. An experienced engineer skilled in inspection procedures and techniques must perform the structural assessment.

Table 5-1. Capability of Each Level of Inspection For Detecting Damage to Waterfront Structures

| Level | Purpose | Detectable Defects | | |
|---|---|---|---|---|
| | | Steel | Concrete | Wood |
| I | General visual to confirm as-built condition and detect severe damage | Extensive corrosion<br><br>Severe mechanical damage | Major spalling and cracking<br><br>Severe reinforcement corrosion<br><br>Broken piles | Major losses of wood<br><br>Broken piles and bracings<br><br>Severe abrasion or marine borer attack |
| II | Detect surface defects normally obscured by marine growth | Moderate mechanical damage<br><br>Major corrosion pitting | Surface cracking and crumbling<br><br>Rust staining<br><br>Exposed rebar and/ prestressing strands | External pile damage due to marine borers<br><br>Splintered piles<br><br>Loss of bolts and fasteners<br><br>Early borer and insect infestation |
| III | Detect hidden and imminent damage | Thickness of material | Location of rebar<br><br>Beginning of corrosion of rebar<br><br>Internal voids<br><br>Change in material strength | Internal damage due to marine borers (internal voids)<br><br>Decrease in material strength |

The time and effort required to carry out the three different levels of inspection are quite different. The time required for any particular level will depend on a number of factors, including visibility, currents, wave action, water depth, severity of marine growth, and the skill and experience of the diver.

Table 5-2 provides a guide for estimating the time required to conduct Level I and Level II inspections. This information is based on:

(1) A water depth of 30 to 40 feet
(2) Visibility of 4 to 6 feet
(3) Warm, calm water
(4) Moderate marine growth (about 2 inches thick)
(5) An experienced diver of average skill

Table 5-2. Production Rate for Surface and Underwater Inspection of Structural Elements

| Structural Element | Inspection of Time Per Structural Element (min) | | | |
|---|---|---|---|---|
| | Level I | | Level II | |
| | Surface | U/W | Surface | U/W |
| 12-in. steel H-pile | 2 | 5 | 15 | 30 |
| 12-in. wide strip of steel sheet pile | 1 | 3 | 8 | 15 |
| 12-in. square concrete pile | 2 | 4 | 12 | 25 |
| 12-in. wide strip of concrete sheet pile | 1 | 3 | 8 | 15 |
| 12-in. diameter timber pile | 2 | 4 | 10 | 20 |
| 12-in. wide strip of timber sheet pile | 1 | 3 | 7 | 15 |

For the Level II inspection it has been assumed that 3 feet of the structural element in the splash zone, 1 foot at mid-depth, and 1 foot at the bottom will be completely cleaned of marine growth. It has also been assumed that the most efficient method of removing marine growth will be used.

Level III inspections depend on the extent of existing damage, the type of inspection techniques, the equipment used (ultrasonic thickness measurements, increment borings, caliper measurements, etc.), and the degree of cleaning required. Therefore, estimates of time for Level III inspections are not included in Table 5-2.

Table 5-3 depicts typical daily rates for swimming-by, cleaning, and taking measurements on piles and linear feet of bulkhead.

Table 5-3. Typical Daily Rates for Underwater Inspection Tasks*

| **Inspection Task** | **Piles/Day** | **Bulkheads in LF/Day** |
|---|---|---|
| Swim-By | 300-600 | 500-1500 |
| Cleaning | 30-70 at 3-15% of each pile | 500-1500 at 50-300 LF intervals |
| Measurement | 50-200 for Wood at 5-15% of each pile<br>30-60 for Steel at 3-10% each<br>30-70 for Concrete 3-15% each | 500-1500 at 50-300 LF intervals |

*Rates can vary widely depending on the effects of many factors such as water visibility, facility size and age, marine growth, and type of construction.

## 5.2 Planning for Inspection

Before starting a facility inspection, all available information about the facility should be obtained. This will usually require a preliminary visit to the facility. The Engineer in Charge should meet with the local staff civil engineer and obtain copies of the facility drawings and

general background about the existing condition of the facility. Any unique features or special problems that may be encountered should be noted. Local information that should be obtained includes:

- Wave action
- Atmospheric temperature range
- Water temperature range
- Tidal range
- Water depths
- Water visibility
- Currents
- Any condition that could have a direct impact on the time needed to perform an inspection, such as amount of biofouling growth on piles or any other condition that would inhibit the performance of an inspection such as ice or seasonal flooding.
- Ship traffic and facility berthing requirements.

The bathymetric and oceanographic data, as well as information on nearby obstructions or activities, will accelerate the planning process and will aid in determining the levels of inspection to be used. The time and effort required to carry out the three different levels of inspection will vary considerably. The factors affecting the time required will include whether the inspection is surface or underwater; the environmental factors mentioned above; and the skill and experience of the inspector.

Information about the local support, equipment, and utilities should be acquired prior to the inspection. Once the information about the facility has been collected, an inspection plan should be developed. The written plan should be like a statement of work containing a scope of work, specifying the sampling criteria (if any work is contracted out, a separate SOW will be required). It should specify responsibilities, tasks, schedules, and equipment to be used.

Of critical importance to the effectiveness of each survey is the proper and adequate selection of the areas to be examined. It is important to select a sufficient number of inspection areas to provide representative information on the overall structure. Making this selection requires an understanding of the facility structural behavior to determine which areas are subjected to maximum stress, fatigue, and impact forces. Knowledge of deterioration and damage theory is also useful. Consequently, the inspection plan must be prepared in cooperation with qualified engineers familiar with the structure. The inspection plan should include the identification of the inspection equipment most appropriate to the specific tasks. For older facilities where little or no data, including drawings, is available, the inspection plan should allot time for developing and/or confirming the structural layout, and confirming whether previously identified repairs have been made.

A suitable scheme should be devised for designating individual piles and other structural members. The inspection team should use the pile numbering/ designation systems available on existing "as-built" drawings where available. Usually, combinations of numbers and letters are used with the number designating the bent and letter indicating the pile within the bent. Legends may be created to represent such things as the degree of deterioration of individual structural members, the level of inspection given to designated portions of a facility, the shape of individual piles, and the type of materials. Pile plans should be prepared for piers showing the lengths, widths, and spacing of bents. The plans must also include the numbering system used in the inspection and in the report, and these must be correlated with existing drawings of the facility. It is desirable to also include design live load data on all pile plans.

## 5.3 Inspection Frequency

The inspection frequency will depend upon whether the inspection is surface or underwater, and the expected rate of deterioration and damage. A typical example requiring more frequent inspection is an area experiencing damage by ships' berthing that results in advanced deterioration to both fender and structural piling. The frequency and level of inspection should, therefore, be closely tied to the historical deterioration rate of the facility. Recommended frequencies are listed in Volume 4 of NAVFAC MO-322, Inspection of Shore Facilities. Research by the Navy on inspection sampling criteria and procedures is published in several technical reports (see Bibliography). Statistical software has been developed which identifies inspection frequencies based on cost when known or estimated structural data is inputted. The frequencies obtained will be unique to the facility's situation. As a general guide, recommended frequencies of inspection for the different types of waterfront structures are as follows:

- All superstructure and piling/sheet piling above the waterline, including the splash and tidal zones (Figure 5-1), should be inspected annually.

- Concrete/steel structural members at the splash/tidal zones and downward should be inspected at least every 6 years. As deterioration is discovered, the level of inspection and frequency needs to be increased accordingly. For steel structures, the age of the structures is a primary factor since the rate of deterioration due to corrosion is fairly constant. Likewise, concrete in a saltwater environment deteriorates chemically with time, especially if cracks are present to allow the seawater to reach the structure's interior.

- Timber members should be inspected at least every three years and, as above, more frequently and intently as deterioration is discovered. In areas where marine animal infestation is known to be a problem, increased inspection frequency is especially important.

Additionally, if it is not feasible to thoroughly inspect all elements of a structure (e.g. underwater inspection of a series of piers containing many piles), selecting an optimum number of structural elements or members is crucial to obtaining accurate information representative of the overall condition of the structure. Development and validation of sampling criteria and procedures have been reported by NFESC (formerly NCEL) in NCEL TN-1762 "Sampling Criteria and Procedures for Inspection of Waterfront Facilities". Statistical sampling techniques using probability theory provide a method for determining condition parameters for the entire population based on information from the sample elements, with a calculated confidence level and precision. Three methods of random sampling have been identified as being applicable to waterfront facilities: simple, systematic, and cluster.

These methods are described in detail in NCEL TN-1762. Excellent correlation between the statistical data and actual conditions is obtainable when inspecting for natural deterioration (e.g. biological or wave action). However, the statistical techniques are not accurate when considering damage due to improper construction and mechanical overloading.

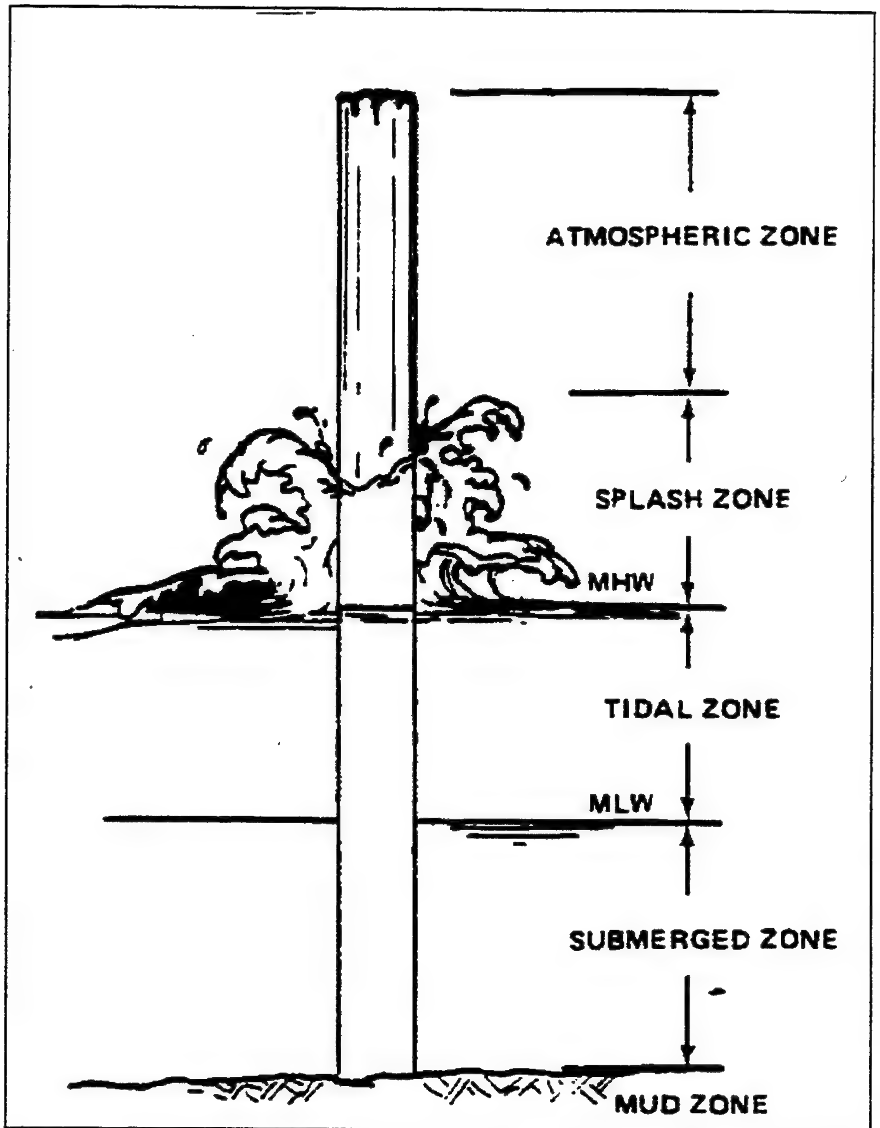

Figure 5-1. Various zones of influence on piling.

### 5.4 Documentation of Inspection

For the results of the inspection to be useful, they must be documented in a clear and concise manner and in accordance with generally understood terminology. Inspectors should maintain daily logs of inspection details including measurement data, locations of observation, and water depths, if relative. Inspection forms and reports should be completed as soon as possible after the inspection has been completed. Standard forms and report formats greatly facilitate the documentation procedure and are essential for comparing the results of the present inspection with past and future inspections. Figure 5-2 is a standard form for reporting the condition of piles; Figure 5-3 is an explanation of the condition ratings for concrete piles used on the form; and Figure 5-4 is an explanation of the condition ratings for timber piles. Steel pile inspection results are usually recorded in terms of remaining metal thickness. It should be noted, however, that the categorization of a defect, i.e., moderate, major, etc., will depend on the water depth. Piles in deeper water, with a long unsupported length, are susceptible to buckling, and loss in strength becomes more critical.

When appropriate, damaged areas should be documented with still photography and closed circuit television. Still photography provides the necessary high definition required for detailed analysis, while video, though having a less sharp image, provides a continuous view of events that can be monitored by surface engineers and recorded for later study. All photographs should be numbered, dated, and labeled with a brief description of the subject. A slate or other designation indicating the subject should appear in the photograph. When color photography is used, a color chart should be attached to the slate to indicate color distortions. Videotapes should be provided with a title and lead-in, describing what is on the tape. The description should include the method of inspection used, the nature and size of the structure being inspected, and any other pertinent information.

A debriefing with the activity personnel, with slides or photographs, should be conducted before leaving the site, and all questions should be resolved.

**PILE INSPECTION RECORD**

| | | |
|---|---|---|
| LOCATION | DATE | DIVERS |
| PIER NAME/NO. | PILE TYPE ☐ BEARING ☐ FENDER ☐ SHEET | PILE MATERIAL ☐ TIMBER ☐ STEEL ☐ REINFORCED CONCRETE |
| WATER DEPTH | TIME OF DAY | TIDE | DEPTH OF DAMAGE FROM DATUM = GUAGE DEPTH - TIDE |

| BENT NO. | PILE NO. | NI | PILE CONDITION | | | | | TYPE DAMAGE | | | GAUGE DEPTH DAMAGE | DIMENSIONS OF DAMAGE | | | COMMENTS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | ND | MN | MD | MJ | SV | MECH. | BIO | FUNC. | | HGT | WIDTH | PENETR | |
| | | | | | | | | | | | | | | | |
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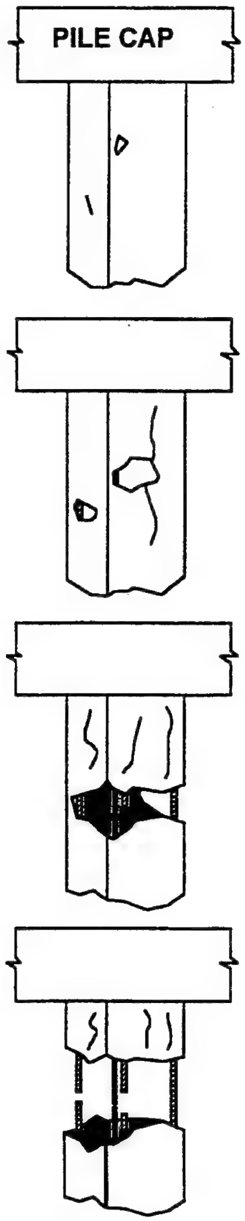
| | | | | | | | | | | | | | | | |

Figure 5-2. Standard pile inspection report form. (See Figure 5-3 and 5-4 for explanation of Ratings.)

CONCRETE PILE
<u>CONDITION RATING</u> <u>EXPLANATION</u>




NI NOT INSPECTED, INACCESSIBLE OR PASSED B

ND NO DEFECTS:
- hairline cracks
- good original surface, hard material

MN MINOR DEFECTS:
- good original surface
- minor cracks or pits
- small chips or popouts
- slight rust stains
- hard material, sound
- corrosion of the wires

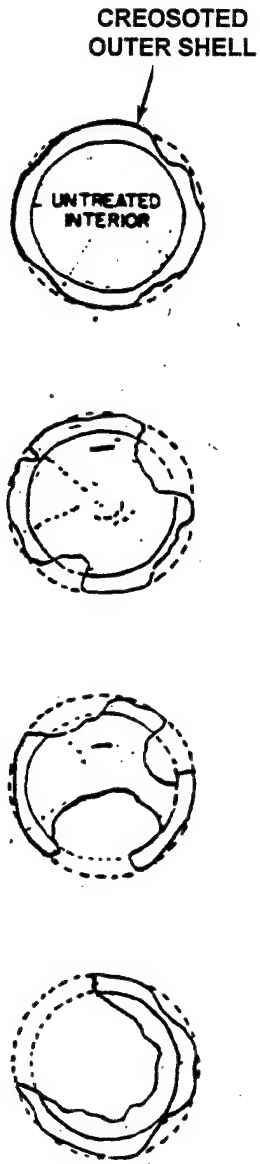
MD MODERATE DEFECTS:
- limited spalling of concrete
- minor corrosion of exposed re-bar
- rust stains along re-bar
- softening of concrete
- reinforcing steel ties exposed
- popouts or impact damage

MJ MAJOR DEFECTS:
- spalling of concrete results in (10-15%) loss
- large spalls six inches or more in width or length
- deep wide cracks along re-bar
- major rust stains along rebar
- wide spread surface disintegration

SV SEVERE DEFECTS:
- exposed rebar with 50% loss of steel section are
- more than 15% loss of concrete

NOTE: Explanation of defect should be placed in the comments column.

Figure 5-3. Explanation of pile condition ratings for concrete piles.




| TIMBER PILE CONDITION RATING | EXPLANATION |
|---|---|
| NI | NOT INSPECTED, INACCESSIBLE OR PASSED BY |
| ND | NO DEFECTS:<br>- Less than 5% lost material<br>- sound surface material<br>- no evidence of borer damage |
| MN | MINOR DEFECTS:<br>- 5% to 15% lost material<br>- sound surface material<br>- no evidence of borer damage<br>- minor abrasion damage |
| MD | MODERATE DEFECTS:<br>- 15% to 45% lost material<br>- significant loss of outer shell material<br>- evidence of borer damage<br>- significant abrasion damage |
| MJ | MAJOR DEFECTS:<br>- 45% to 75% lost material<br>- significant loss of outer shell and interior material<br>- evidence of severe borer damage<br>- severe abrasion damage |
| SV | SEVERE DEFECTS:<br>- more than 75% lost material<br>- no remaining structural strength<br>- severe borer damage |

NOTE: Explanation of defect should be entered in the comments column.

Figure 5-4. Explanation of pile condition ratings for timber piles.

## 5.5 Equipment and Tools

Following is a general discussion of tools required for the underwater inspection process. Each of the sections discussing the inspection of particular structure types will address the particular equipment and tools required to inspect that structure.

**5.5.1 Surface Cleaning Tools**. To perform a thorough inspection, the marine growth on the structure must be removed. This can be done by various means, depending on surface support. For small sample areas, wire brushes, probes, and scrapers may be adequate. For larger

areas or more detailed inspections underwater, a hydraulic grinder with barnacle buster attachment, or high-pressure water jet gun, may be used. Exercise care to prevent damage to the preservative-treated layers of timber or deteriorating surfaces of concrete.

**5.5.2 Inspection Tools**. Inspection tools and equipment include:

(a) Hand-held tools such as portable flashlights, rulers, and tape measures for documenting areas; small or large hammers or pick-axes for performing soundings of the structural member; calipers and scales for determining thickness of steel flanges, webs, and plates, or diameters of piling; increment borer and T-handles for extracting core samples from timbers; and chipping tools for prodding the surface of the concrete to determine the depth of deterioration.

(b) Mechanical devices including a Schmidt test hammer for measuring concrete surface hardness and rotary coring equipment for taking core samples from concrete structures.

(c) Electrical equipment such as an underwater voltmeter for determining the level of cathodic protection on steel structures and underwater sonic and ultrasonic equipment for detecting voids in timbers or concrete and thickness of structural steel. Underwater magnetic particle testing to locate and define surface discontinuities in magnetic materials.

**5.5.3 Recording Tools**. Recording tools and equipment are required to provide a complete documentation of the condition of the structure. Simple tools such as clipboards, forms, and cassette recorders for above water inspections, or a Plexiglas slate and grease pencil for underwater inspection, provide the basic documentation tools. More in-depth documentation may be obtained with above-water or underwater photography using colored still-frame cameras or colored video, or closed-circuit television. The latter may be very valuable in expediting major underwater inspections. For underwater inspections in turbid water, a clear-water box may be fitted to the lens of the photographic or video equipment to improve visibility between the lens and surface to be inspected.

## 6.0 STEEL STRUCTURES

### 6.1 Types of Steel Structures

Structural steel is used in most metal waterfront structures because it is strong, readily available, easily fabricated into any shape, and not excessively costly. In marine applications, steel has many uses as a construction material. Steel piles, either H-piles or pipe piles, are used as support members for open piers, wharves, and other waterfront structures and facilities. Steel sheet piling is used primarily as a retaining wall structure for bulkheads used in the support of piers, wharves, drydocks, and quaywalls as well as near-water, earth-retaining structures. Fabricated structural steel members, whether tubular, plate, or other shapes, are used to construct undersea support towers for testing ranges, instrument arrays, and operation support platforms.

## 6.2 Deterioration of Steel Structures

There are six major types and causes of steel structure deterioration in the marine environment:

- Corrosion
- Abrasion
- Loosening of structural connections
- Fatigue
- Overloading
- Loss of foundation material

**6.2.1 Corrosion.** Corrosion is the principal cause of deterioration of steel waterfront structures. Corrosion of steel is an electrochemical process that converts the steel into iron oxides. These iron oxides are easily recognized by their reddish-brown color and are commonly called rust. The rust may remain in place in the form of an encrustation or may naturally fall away or be removed by wave action or abrasion. The corroded surfaces are usually irregular and in some cases the attack in localized areas will be much greater than in other areas resulting in pitting. Over a period of time, unchecked corrosion will reduce the structural integrity of steel components of waterfront structures

On bare unprotected steel pilings, corrosion is often most severe just above the high tide line, with another zone of severe attack just below the low tide line, as shown in Figure 6-1. Figures 6-2 and 6-3 illustrate typical corrosion on steel H-piling and sheet piling.

Submerged steel is also subject to galvanic corrosion. Galvanic corrosion occurs when two dissimilar metals are in electrical contact and both are submerged. An electrical current then flows between the two metals causing one of them to corrode rapidly. The composition of the metals, the exposed area, and the electrical conductivity of the liquid govern the speed of the attack. Salt water is an excellent electrical conductor, and galvanic corrosion resulting from dissimilar metals in contact is a significant problem on waterfront structures. The possibility of galvanic corrosion must be considered whenever dissimilar metals are used in marine structures.

An effect similar to galvanic corrosion can occur when there is a difference in environment between different areas on a single metal. When oxygen levels are limited, by reduced free access to freshly oxygenated water, in areas such as the threads of a bolt or between structural members, the difference in oxygen content can cause an electrical current to flow between the areas of high and low oxygen content. This electrical current results in accelerated corrosion. This effect is particularly severe on stainless steels and aluminum alloys. A difference in environment with resulting accelerated corrosion can also be created when a surface is partially covered by an electrically conductive material such as concrete.

Corrosion can also be accelerated by the action of bacteria. In the absence of oxygen, sulfate-reducing bacteria can grow, and the sulfides they produce can cause rapid attack on steel. Areas such as the inside of pipe piles that are not filled with concrete can become depleted in oxygen and are frequently the site of sulfate-reducing bacteria attack. Some bottom sediments are oxygen deficient and sulfate-reducing bacterial attack can occur on buried steel in such sediments. Sulfate-reducing bacteria attack is more likely to occur in polluted harbors but can occur whenever oxygen is depleted. The presence of sulfate-reducing bacteria can often be detected by the odor of rotten eggs produced by the bacteria. As sulfate-reducing bacteria attack usually occurs on the inside of pipe piles or below the mudline, it is especially difficult to locate and repair open end pipe piles which are not concrete filled and typically used on offshore and

waterfront structures. Experience indicates, however, that the insides are generally not subject to corrosion.

Finally, corrosion can be initiated by the presence of stray currents, e.g., from nearby electrical power lines.

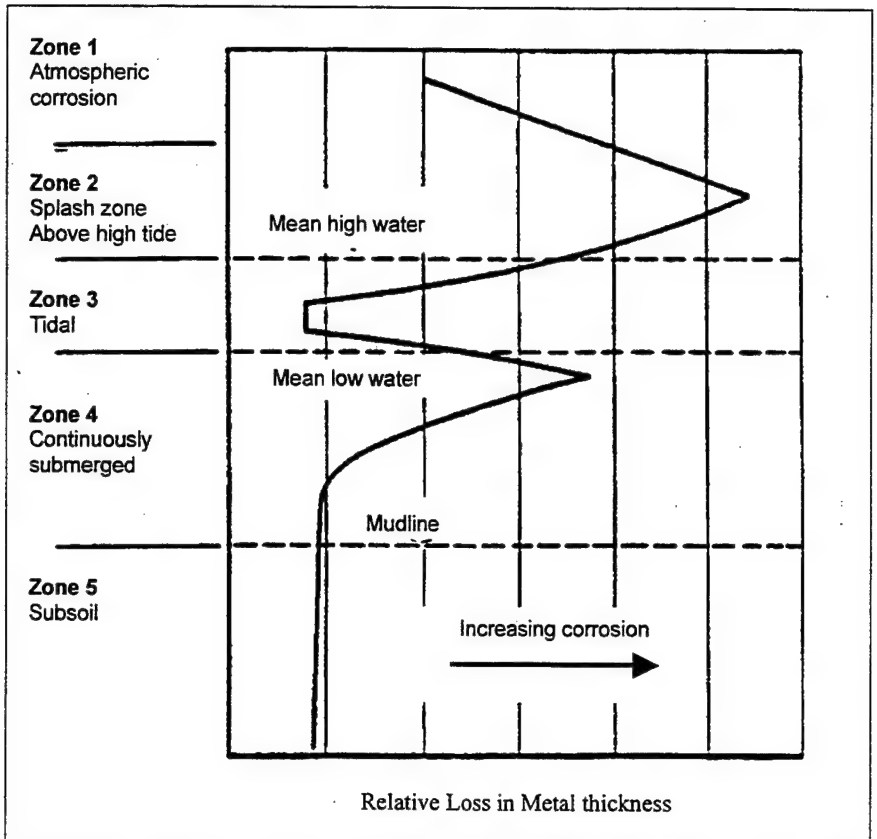



Figure 6-1. Severe corrosion zones, just above high tide line and just below low tide line.

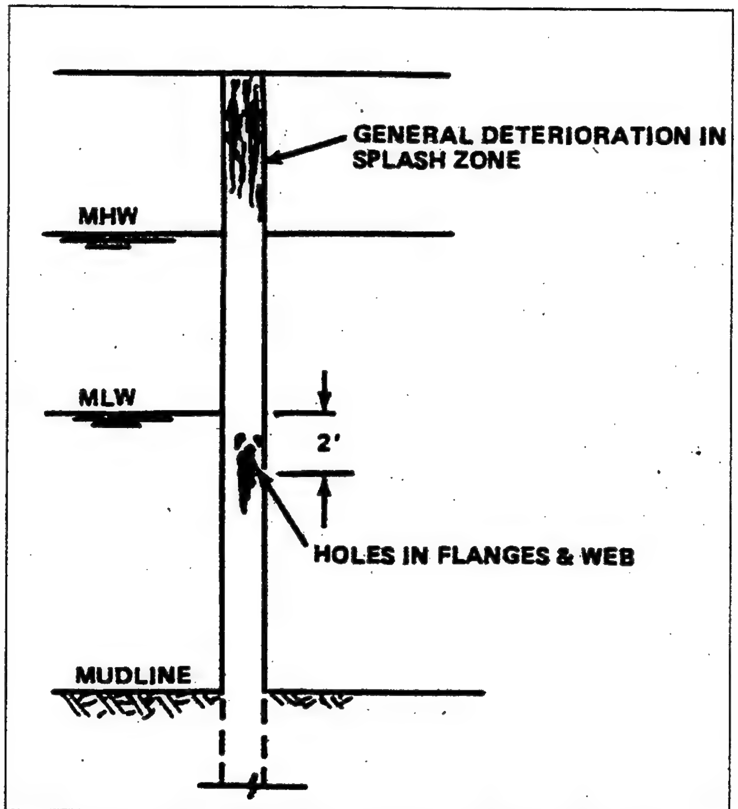



Figure 6-2. Typical corrosion on Steel H-piling.

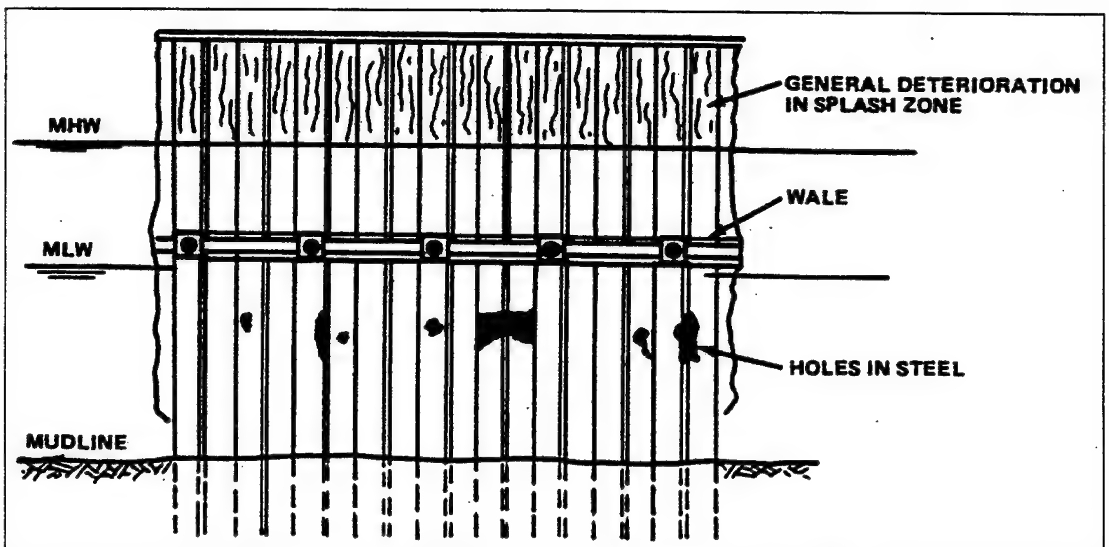



Figure 6-3. Typical corrosion on steel sheet piling.

**6.2.2 Abrasion**. Abrasion of steel structures can generally be recognized by a worn, smooth, polished appearance of the surface. Abrasion is caused by continual rubbing of adjacent moving steel surfaces, or by the exposure of structural components to wave action in areas of sandy bottom. Steel sheet piling and pile-supported structures are particularly susceptible to sand abrasion in exposed locations. Abrasion of steel structures is a problem because it removes both protective coatings and protective layers of corrosion products, thus accelerating corrosion.

**6.2.3 Structural Connection Loosening.** Structural connections joined together by rivets or bolts have a tendency to work loose over an extended period of time. Connection loosening can result from impact loading of the type imparted by a vessel striking a pier or wharf fender system. Wave action and reciprocating machinery mounted on or below pier or wharf decks are other sources of possible connection loosening. Corrosion of bolts, rivets, nuts, washers, and holes can also contribute to connection loosening. Loosening of connections will tend to produce misalignment in mating surfaces, which, in turn, can result in distortion and stress concentrations in framing members.

**6.2.4 Fatigue Failure.** Fatigue failure results in the fracture of structural members as a consequence of repeated high loading. Fatigue distress can be recognized by a series of small hairline fractures perpendicular to the line of stress in the member. Tubular connections of offshore platforms are particularly susceptible to fatigue failure. Fatigue cracks are difficult to locate. Since fatigue cracks represent an extremely dangerous condition in steel marine structures, extreme care must undertaken when inspecting structural members subjected to repetitive loading, particularly high wave loading.

**6.2.5 Overloading.** Steel structural elements are sensitive to impact damage from berthing vessels and other types of accidental overloading. Impact or collision damage can generally be recognized by the appearance of local distortion (deformation) of the damaged member. This damage is generally characterized by a sharp crimp or a warped surface as illustrated in Figure 6-4. Compression overloading damage of a steel pile is illustrated in Figure 6-5.

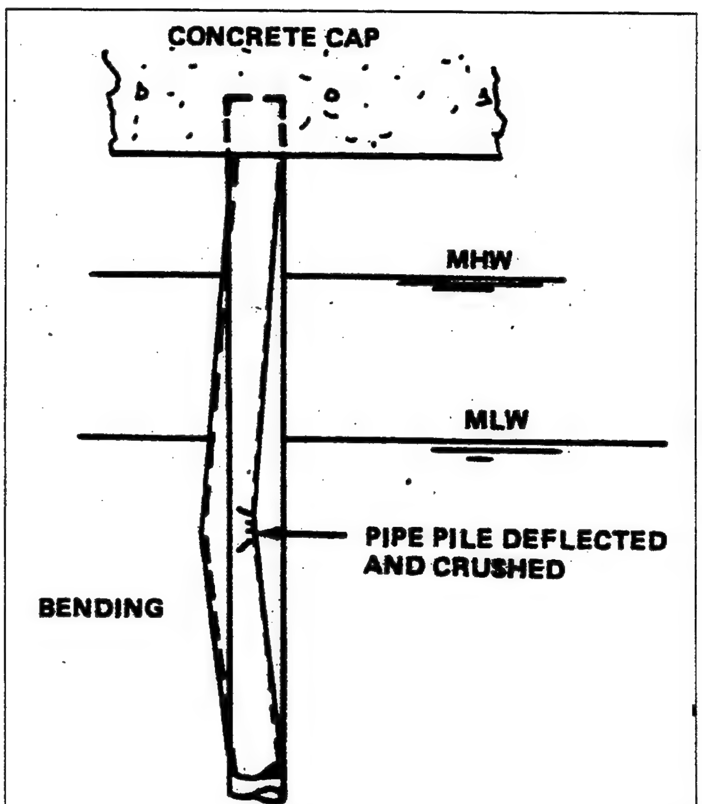


Figure 6-4. Overloading damage due to impact or collision.

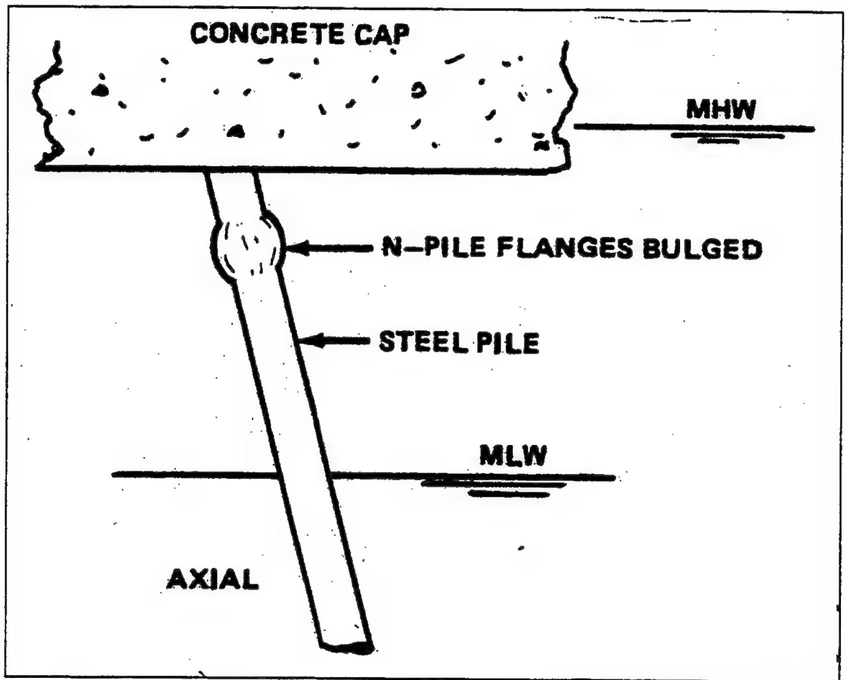


Figure 6-5. Overloading damage due to compression.

**6.2.6 Foundation Deterioration**. Loss of foundation material from around steel piles leads to accelerated corrosion and loss of column strength of the piles. The loss of foundation material is usually caused by the scouring of material from around the piles. Scouring can be caused by an increase in the velocity of the water passing by the piles or a change in the current's direction. If the piles thus exposed are not protected, eventual collapse of the structure is possible. A loss of foundation material in front of a sheet pile bulkhead may cause kick-out of the toe of the wall and result in total failure.

## 6.3 Typical Inspection Procedure

**6.3.1 Surface Inspections.** Generally, visual inspections will allow detection and documentation of most forms of deterioration of steel structures. In the event that more detailed NDT techniques may be required under a Level III inspection, a plan and sampling techniques need to be developed and tailored to the specific areas of concern.

Some types of corrosion may not be detected by visual inspections. For example, inside steel pipe piling, anaerobic bacterial corrosion caused by sulfate-reducing bacteria is especially difficult to detect by visual inspection. Fatigue distress can be recognized by a series of small hairline fractures perpendicular to the line of stress but these are difficult to locate by visual inspection. This type of problem, however, is more prevalent to offshore platforms with welded structural connections than to standard piers and wharves.

Cathodic protection systems need to be closely monitored both visually and electrically for signs of loss of anodes, wear of anodes, disconnected wires, damaged anode suspension systems, and/or low voltage.

**6.3.2 Underwater Inspection.** Underwater Inspection of a steel waterfront structure should proceed as outlined in Table 6-1.

Table 6-1. Steel Structures Underwater Inspection Checklist.

| Checkpoint | Description |
|---|---|
| 1 | Start the inspection at the splash/tidal zones and proceed to about 3 feet below the MLW. This is where most mechanical and corrosion damage is normally found. |
| 2 | Clean all marine growth from a 1-foot square section of pile (a larger areas, if inspecting sheeting) and visually inspect for rust, scale, and holes. |
| 3 | If the structure has a cathodic protection system, check the cleared underwater area with an underwater voltmeter, as shown in Figure 3-22, or use a portable reference electrode and portable reference cell, as shown in Figure 3-23, to determine the effectiveness of the cathodic protection system. Acceptance levels for cathodic protection are -0.80 to -0.90 volt when compared to a silver/silver chloride reference electrode. |
| 4 | Sound the surface with a hammer to detect any scaled steel or hollow areas. |
| 5 | Inspect holes in steel sheeting for loss of backfill material through the opening and subsidence of adjacent ground surface. |
| 6 | Descend, visually inspecting the structure and sounding with a hammer where there is minimal marine growth. |
| 7 | At the bottom, record the water depth, using a wrist gauge, on a Plexiglass slate with a grease pencil, or communicate the information to topside personnel. |
| 8 | Record other visual observations, such as coating condition (peeling, blistering, erosion). Closely inspect splices for loss of weld materials and looseness. |
| 9 | Record the condition of cathodic protection equipment (broken or corroded conduits, loose wires, consumed or lost anodes). |
| 10 | Record the extent and type of corrosion, structural damage, or any other significant observations, using calipers and scales to measure the thickness of steel flanges, webs, and plates, and ultrasonic meters to measure the thickness of steel pipe piles and sheet piling. |
| 11 | Return to the surface and immediately record the observation data in the inspection log, or communicate data to topside personnel. |
| 12 | Where more sophisticated means are required to evaluate the condition of steel piling:<br>• Ultrasonic inspection is available for thickness measurements. Pay particular attention to ensure that the areas are clear of all marine growth and scale. Use of ultra-sonic equipment in areas with corrosion pitting can result in erroneous thickness measurements.<br>• Magnetic particle inspection may be used, particularly on welded connections, to detect cracks and small defects. |
| Exposed Area Under Pier or Along Wharf | |
| 1 | Inspect for structural damage, rust, scale, and holes. |
| 2 | Sound the surface with a hammer to detect any scaled steel or hollow areas. |
| 3 | Inspect holes in steel sheeting for loss of backfill material through the opening and subsidence of adjacent ground surface. |

## 6.4 Equipment and Tools Required

To ensure a thorough inspection, the area must be cleared of all marine growth. This can be done by various means, depending on surface support. A high-pressure water jet is the most effective method for clearing marine growth sufficiently for visual inspection. Hand tools, such as wire brushes and scrapers are sufficient for smaller jobs. Sounding of the structure can be done with a small hammer or pickax. Inspection of the structure requires some type of underwater data recording device, such as a grease pencil/slate, or hardwire communications with topside personnel. Calipers and scales are used to determine thicknesses of steel flanges, webs, and plates. A portable reference electrode or an underwater voltmeter is used to determine the effectiveness of cathodic protection on steel structures. Table 6-2 gives voltages measured by the underwater voltmeter for steel structures. Visual documentation may be desirable for illustrating problem areas to others who did not physically make the inspection dive. Using an underwater camera or television system can do this.

Table 6-2. Underwater Voltmeter Values for Steel Structures.

| Voltage Measured (V) | Description |
|---|---|
| 0.0 to –0.7 | Steel is cathodically unprotected. The rate of corrosion depends on the effectiveness of paint or tar coatings, marine growth, and local water chemistry and water currents. On some structures, the hard layer of marine growth may provide some protection. The closer to 0 volts the more active is the corrosion potential. **Note**: -0.6 volt is the potential of bare, unprotected steel in seawater. |
| -0.7 to –0.82 | The steel is partially protected. |
| -0.83 to –1.1 | The steel is adequately protected. Cathodic protection systems are working effectively. |
| -1.1 or higher negative values | The steel is "overprotected." **Note**: Under some circumstances, the metal surface can be made more brittle when overprotected. Surface coatings may be damaged or "lifted off" by the excess formation of hydrogen bubbles. |

**6.4.1 Ultrasonic Inspection**. Ultrasonic thickness measurement equipment is available for inspecting steel structures. Thickness measurements are obtained because certain types of ultrasonic waves travel at a constant speed through a material, because they travel in straight lines, and because a portion of the wave is reflected when it meets an interface. The difference in time between the detection of the front surface and back surface echoes is correlated to the thickness of the material. Ultrasonic thickness measurements require a thorough removal of marine growth and scale and can be unreliable if the surface on which the instrument is placed is heavily pitted. Adequate training and experience are required to obtain readings and to evaluate the measurements made with this equipment.

**6.4.2 Underwater Magnetic Particle Testing.** Underwater magnetic particle testing (UWMT) is a nondestructive method for locating and defining surface discontinuities (such as cracks) in magnetic materials underwater. Its principle of operation is that magnetic particles are attracted to flux leakages at the surface of magnetized materials and form indications of discontinuities located either at or just below the surface.

In operation, the magnetic material or item of interest is magnetized using an electromagnetic yoke specially designed for underwater use. Wherever surface discontinuities exist within the yoke's field of influence, magnetic flux will "leak" from the surface of the part. A slurry of magnetic particles are attracted to, and aligned with, the leaking magnetic flux. The particles are brightly colored and form a visible indication corresponding to the location of discontinuities at or very near the part's surface.

- **UWMT Applications** - Underwater magnetic particle testing is used primarily as a quality assurance tool to support underwater welding on ship structures. It can also be used to inspect hulls or other magnetic components for surface discontinuities such as cracks and lack of fusion in welds. UWMT can be used to define the true length (and locate the true ends) of discontinuities detected visually and to help determine where corrective measures (e.g., stop drilling) should be applied.

- **UWMT Limitations** - As with any inspection method, UWMT has some limitations. These include:

(1) Underwater magnetic particle testing has limited subsurface capability. It is considered to be strictly a method for detecting and measuring surface discontinuities. It is not an approved method for detection of subsurface discontinuities.

(2) The adequacy of inspection with UWMT (as with most nondestructive test methods) is largely a function of the operator's knowledge and skill. Inspections with UWMT are to be performed only by personnel trained and specifically certified in UWMT.

(3) UWMT is limited to ferromagnetic materials, which include most steels. For most applications, a simple check with a magnet is sufficient to determine suitability for UWMT.

- **Certification Requirements for UWMT** - Personnel performing UWMT require certification in UWMT. Certification can only be obtained by training and examination. Divers may be trained by a certified UWMT examiner. Training can be done at either the diver's activity or at the agent's facility.

- **Specific Preparation Requirements for UWMT** - Preparation for conducting UWMT entails assembling all necessary material and personnel required to safely satisfy the plan requirements. Divers must determine that the water current will not affect the application of magnetic particles where UWMT is to be conducted. Water currents greater than 1 knot make it difficult to perform UWMT. Divers must also determine that the underwater visibility is adequate for the interpretation of the test results.

The following paragraphs describe inspection equipment required to conduct UWMT. Inspection equipment specific to UWMT, along with general surface preparation and recording equipment are:

- **Inspection Equipment**

  Electronic yoke with power cable
  Ground fault interrupter (GFI)
  White light source
  U/W-1 magnetic particles with applicator
  Magnetic field indicator

- **Surface Preparation and Restoring Equipment**

  Diver staging (optional)
  Hydraulic or pneumatic hand-held grinder or high-pressure water jet
  Anti-corrosion coating (epoxy)

- **Recording Equipment**

  Stereo and/or still camera (optional)
  Video and monitor system (optional)
  Measuring devices (tape or rule)
  Plexiglas writing slate, grease pencil, arrow punch

UWMT equipment includes:

- **Electromagnetic Yoke** - An electromagnetic yoke is used to induce a magnetic field in the material. The articulation of the yoke's legs allows any pole spacing between 2 inches and 8 inches, and the yoke can accommodate plate offsets of up to 6 inches and joint angles from about 45 degrees to 270 degrees. To minimize the risk of electric shock, no controls are on the yoke. A topside operator energizes the yoke.

- **Power Cable** - A two-conductor power cable with a braided external ground and a protective jacket, delivers power to the electromagnetic yoke. Typical cables are 250 feet long to permit operation in the majority of locations accessible by a surface-supported diver. The external ground braid generates a ground fault whenever the cable is cut; the power conductors cannot be reached except by first penetrating the ground braid.

- **Ground Fault Interrupter (GFI)** - The diver's primary protection from electric shock is an approved isolation transformer/GFI device. The GFI interrupts power when it senses a drop in the resistance between the isolated system power leg and the power supply ground.

- **White Light Source** - A primary illumination source is a Remote Ocean Systems model TUBE-LIGHT.

- **Magnetic Particles** - Magnetic particles are finely divided ferromagnetic particles having a low magnetic retentivity and a high permeability. They are dyed pink to be visible under normal lighting. The particles are mixed with wetting agents and corrosion inhibitors to enhance their underwater performance.

- **Magnetic Particle Applicator** - The magnetic particle applicator is a reservoir of magnetic particles and water that the diver uses to deliver magnetic particles to the surface of the inspection area. A simple and effective magnetic particle applicator is a plastic squeeze bottle that contains a marble-sized object to aid in mixing.

- **Magnetic Field Indicator** - The magnetic field indicator is a small device with crack-like discontinuities on its face. The indicator is used to determine if the inspection site has adequate magnetic flux. The diver places the indicator at the inspection site, topside energizes the yoke, and the diver delivers the particles. A clearly visible accumulation of particles (indications) should then form along the crack-like discontinuities on the pie-shaped magnetic field indicator. In the pie-shaped gauge, the crack-like discontinuities are furnace-brazed joints between adjacent steel wedges. Though a simple test to measure the magnetic field strength inside the material being inspected is unknown, it is assumed that an adequate field just outside the material signifies an adequate field inside as well.

## 7.0 COMPLIANT MOORINGS

### 7.1 Types of Compliant Moorings

Compliant moorings allow ships to maintain either a fixed or semi-fixed position. A compliant mooring in common use is the riser free-swinging mooring as shown in Figure 7-1. This type of mooring achieves its holding power through anchors embedded in the seafloor. These anchors are connected to ground leg chains that meet at the ground ring which, in turn, is connected to a riser-type buoy by a riser chain.

Another compliant mooring is the nonriser free-swinging mooring, illustrated in Figure 7-2. This configuration uses anchors embedded in the seafloor that are individually connected to a telephone buoy. Both riser and nonriser moorings require a good deal of unobstructed water surface to allow the ship to rotate around the mooring according to changes in wind and current direction.

A spread mooring eliminates the tendency for the buoy to rotate. This type of mooring uses two or more mooring buoys, usually in conjunction with the ship's anchors, to hold the ship in a fixed position. The most commonly used types of buoys are illustrated in 7-3.

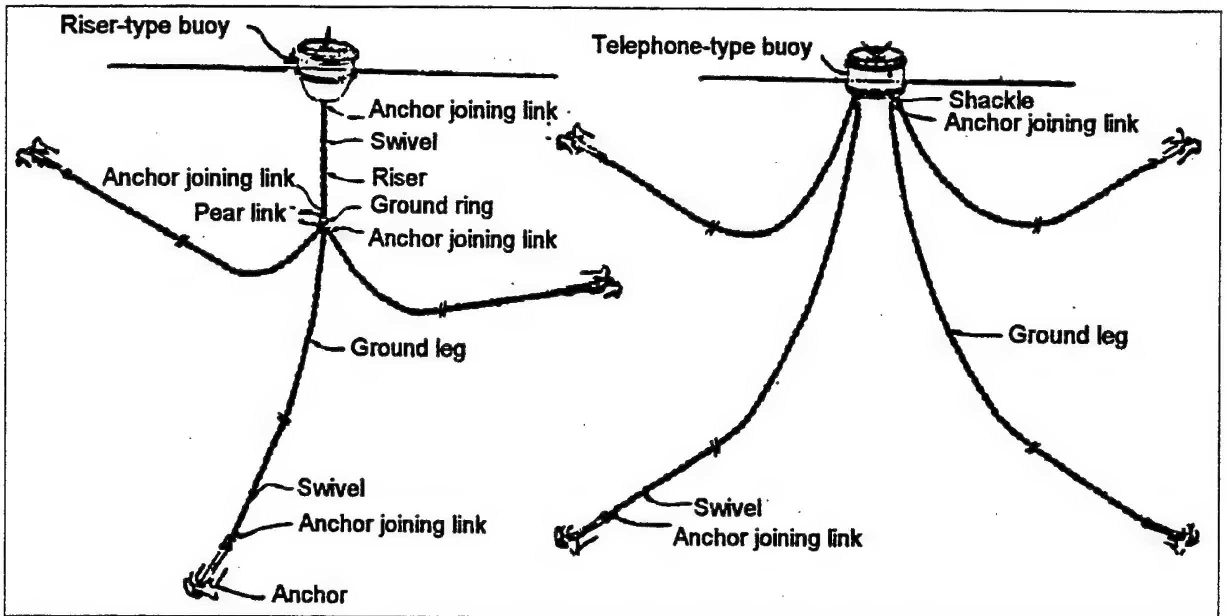



Figure 7-1. Free swinging mooring. Figure 7-2. Non-riser free swinging mooring.

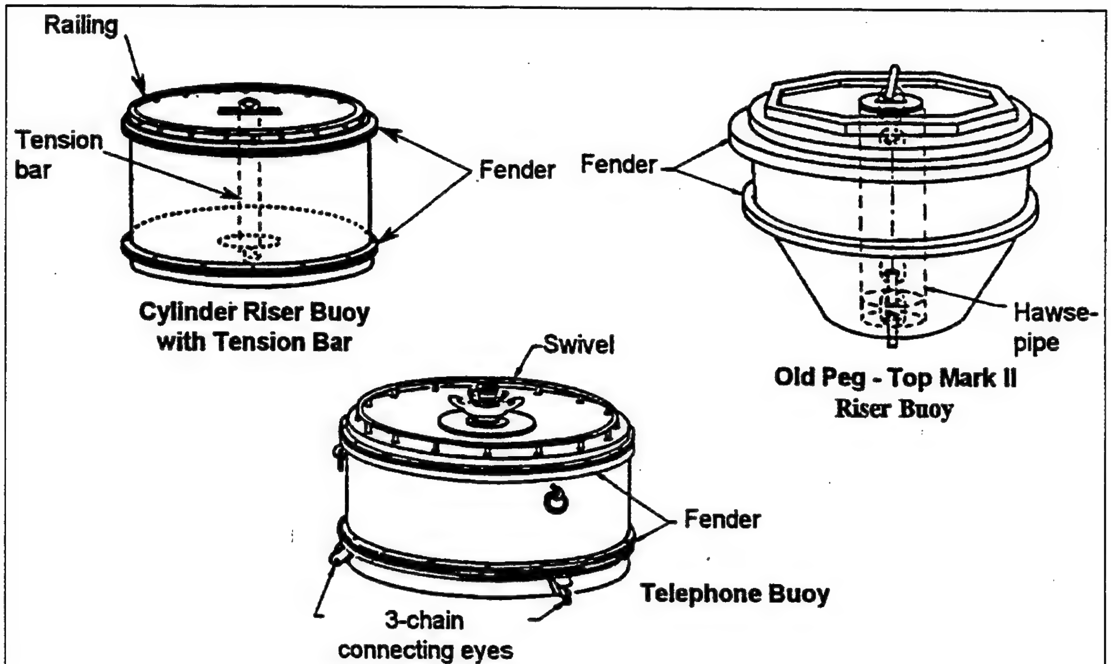



Figure 7.3. Commonly used buoys.

### 7.2 Compliant Mooring Anchors

Not being able to comment on typical mooring anchors that may be found in marine terminal facilities, some common types of anchors used in Navy compliant moorings are:

- **Drag Embedment Anchors**. Of the common drag embedment anchor types found in Navy moorings, the NAVMOOR anchor is typically found in high capacity moorings (Class C - 100,000 pounds and above); while the stabilized Stockless anchor is used in moderate to low

capacity moorings (Class C or below). Stockless anchor flukes will be fixed in the fully opened position for mud soil applications.

When mooring load in a ground leg exceeds the capacity of a single anchor, multiple anchors are used side by side or in tandem (piggyback). In mud seafloors, the piggyback and connecting chain will be buried. However, in hard clay and sand seafloors, the connecting chain and hardware will be exposed and available for inspection.

- **Wedge Anchor (Pearl Harbor Anchor).** This inexpensive anchor may be found in low capacity and short-scope moorings in single and multiple anchor ground legs. Both primary and tandem chain connections and tandem chain often are exposed on the bottom and thus are available for inspection. For rock seafloors, this anchor may be fitted with steel digging plates on the front anchor face, which may be worn or damaged and should be inspected.

- **Direct Embedment Anchors.** Two direct embedment anchor types, which are characterized by their method of installation, are used in Navy compliant moorings. They are the propellant-embedded anchor (PEA) and the pile-driven plate anchor (PDP). The PEA with a wire downhaul cable is fired into the seafloor from a gun (harpoon-like). The wire cable is particularly susceptible to wear at the soil, coral, or rock interface. The PDP is driven into the seafloor by a pile driver and follower assembly. A chain or chain and wire combination can be used with the PDP, but the chain segment always occurs at the soil or coral interface to reduce wear potential. The seafloor interface segment of wire or chain connected to all direct embedment anchors should be inspected.

- **Pile Anchors.** This anchor is used often with bow ground legs of Mediterranean moorings. Its use should be limited in the future due to expanded use of driven plate anchors. The pile anchor to chain connection is usually located below the mudline. Inspection of this connection can be accomplished using a suction dredge to uncover the connection. Inspection is only necessary if severe chain wear/corrosion is evident at the mudline.

## 7.3 Deterioration of Compliant Moorings

Deterioration of compliant moorings is primarily due to corrosion and wear losses on the mooring chains, fittings, and anchors. Compliant moorings are affected by loss of buoyancy due to accumulations of fouling organisms. Compliant moorings are also affected by both electrochemical corrosion and a form of corrosion known as fretting. Fretting is the combined effect of corrosion and ordinary wear. Fretting and ordinary wear is caused by the relative movement between interconnecting links and fittings under the influence of waves, currents, tidal variations and action from the motion of the buoys. Corrosion and wear of metal components is observed as pitting, holes, cracks, fissures, loose or missing bolts or rivets, and reduction of chain wire diameter. Reduction of chain wire diameter is often greatest at interlink connections.

**7.3.1 Chain-Link Measurements.** One significant parameter used to evaluate the condition of a mooring is the chain wire diameter. A selective sampling of wire diameter of chain links and connecting hardware is taken to determine the amount of deterioration due to corrosion and wear.

- "Single-link" measurements are taken where chain is slack.

- "Double-link" measurements are taken where two links connect under tension.

Chain links and other chain components that measure greater than 90 percent of original wire diameter are considered to be in "good" condition. Measurement between 80 and 90 percent of original diameter is considered "fair" condition and is cause for the mooring to be downgraded in classification. Measurement less than 80 percent is considered an indication of "poor" condition and is cause for the mooring to be declared unsatisfactory for use.

## 7.4 Typical Inspection Procedure

Inspecting a compliant mooring should be in accordance with the following checklist. The information should be recorded on a standard compliant mooring underwater inspection report.

### 7.4.1 Inspection Checklist

***1. BUOY UPPER PORTION***

***a. Overall Condition***

- Record buoy type (drum, peg top, etc.).

- Measure and record buoy diameter and freeboard (waterline to top of buoy).

- Buoy overall condition: report any visible damage or listing. If the buoy is listing, determine which inner compartment has water in it.

- Report the color and markings. Ensure that the identification number on the buoy is the same as that depicted on navigation charts; if not, report it.

***b. Fiberglass Coating***

- Report hull dents or separation of the fiberglass from the buoy. The metal could be indented and the fiberglass could look undamaged.

- Report peeling or loose seams or edges. Fiberglass will often fail there first.

- Report any rust bleeding. This indicates trapped moisture between the fiberglass and the buoy's hull.

- Report blisters, bubbles, cracks, checking, or glazing that may be hidden under paint.

***c. Painted Surfaces***

- Report spalling, cracking, peeling, and blisters.

- Report lack of full paint coverage of the buoy or paint discoloration due to chemical reactions or rust bleeding.

MOORING NO. ____________ CLASS. ____________ LOCATION ______________ LAT. ______________ LONG. ________________

ANCHOR TYPES/SIZE: ________________ BUOY TYPE: __________ BUOY DIAM: __________ BUOY COLOR/MARKINGS: ________________

BOTTOM TYPE: ☐ SAND ☐ MUD ☐ CLAY ☐ CORAL ☐ ROCK VISIBILITY __________ D = DEPTH NI = NOT INSPECTED, INACCESSIBLE

| COMPONENTS | | NI | NEW | CONDITION | | | | | | | COMMENT/DESCRIPTION |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | SINGLE LINK (%) | | | DOUBLE LINK (%) | | | D | |
| | | | | 90+ | 80+ | 80- | 90+ | 80+ | 80- | | |
| **Chain Measurements** | | | | | | | | | | | |
| Riser | Near Buoy | | | | | | | | | | |
| | Middle | | | | | | | | | | |
| | Near Grd Rg | | | | | | | | | | |
| **Ground Ring** | | | | | | | | | | | |
| Ground Leg No. A | Upper End | | | | | | | | | | |
| | Middle | | | | | | | | | | |
| | Enters Bottom | | | | | | | | | | |
| Ground Leg No. B | Upper End | | | | | | | | | | |
| | Middle | | | | | | | | | | |
| | Enters Bottom | | | | | | | | | | |
| Ground Leg No. C | Upper End | | | | | | | | | | |
| | Middle | | | | | | | | | | |
| | Enters Bottom | | | | | | | | | | |
| Ground Leg No. D | Upper End | | | | | | | | | | |
| | Middle | | | | | | | | | | |
| | Enters Bottom | | | | | | | | | | |

| Component | Condition | Comments/Description |
|---|---|---|
| Overall Buoy Condition | | |
| Buoy Fiberglass Coatings | | |
| Paint Condition | | |

Date: ________________ Engineer in Charge: ______________________ Divers: ______________________

(Continued on back)

Figure 7-4. Standard mooring inspection report form.

| Component | Condition | Comments/Description |
|---|---|---|
| Hull Condition | | |
| Top Jewelery | | |
| Top Fenders/Chaffing Strips | | |
| Top Manhole | | |
| Top Tension Bar | | |
| Top Hawsepipe | | |
| Lower Fender | | |
| Lower Hull | | |
| Lower Hawsepipe | | |
| Bottom Jewelery | | |
| Buoy Cathodic Protection | | |
| Bottom Tension Bar | | |
| Joining Hardware | | |
| Swivel | | |
| Top Riser Joint Hardware | | |
| Bottom Riser Joint Hardware | | |
| Ground Ring | | |
| Ground Leg A Joint Hardware | | |
| Ground Leg B Joint Hardware | | |
| Ground Leg C Joint Hardware | | |
| Ground Leg D Joint Hardware | | |
| Ground Leg Swivels | | |
| Chain Anodes | | |
| Visible Anchors | | |
| Anchor Hardware | | |

Figure 7-4. Standard mooring inspection report form (continued).

*d. Top Jewelry*

- Identify each component and prepare a sketch depicting the location of each within the top jewelry.

- Report any wear or corrosion of jewelry components.

*e. Fenders/Chafing Strips*

- Record the number and location of each.

- Record the method of fender/chafing strip attachment.

- Check for and report any loose, rusted, or broken attachments or bolts.

- Check the welds securing the fender/chafing strip mounting brackets to the buoy hull and report any cracks or separation of the welding material from the parent metal.

- Ensure that drainage holes through the chafing strips are open and not clogged with debris.

**(1) Timber**

- Report any splintering, dry rot, worm/borer holes, or broken sections.

- Record paint type and condition.

**(2) Rubber**

- Check for and report any rubber brittleness or cracking.

- Record any tears, rips, or missing sections.

**(3) Steel Pipe Chafing Rail**

- Record pipe rail diameter and height above the deck.

- Check for rust and a secure attachment at the base of the stanchions and rust on the underside of each horizontal rail.

- Record any damage, i.e, dents, fractures, or loose parts on which a line may foul.

*f. Manhole Covers*

- Report the number, size, and location of each manhole.

- Report rusting of the covers or bolts. The edges of the covers may show a "delamination" of the steel.

- Check for and report any loose or missing bolts.

- On fiberglass-coated buoys, report whether the manhole covers are fiberglassed or not.

*g. Tension Bar*

- Check eye for wear and measure its diameter with calipers.

- Measure steel bar thickness.

- Check base plate for cracks, warping, or other damage.

- Record plate thickness.

*h. Hawsepipe*

- Measure and record wire diameter and condition of chain held in place by the retaining plate.

- Check for and report bell mouth rusting or wear.

*2. BUOY LOWER PORTION (UNDERWATER)*

*a. Lower Fender*

- Record fender material.

- If timber, report any splintering or broken/missing sections.

- If rubber, report any tears, rips, or missing sections.

- If visible, record fender attachment method.

- Check for and report any rusted, loose, or broken attachments or bolts.

*b. Buoy Bottom*

- Record marine growth thickness.

- If there is no appreciable marine growth, check and record the type and condition of the protective coating (paint or fiberglass).

- Report any dents or other bottom hull damage.

### c. *Tension Bar*

- Check lower tension bar eye for wear and measure its wire diameter with calipers.

- Check retaining plate and report any observed wear or warping.

### d. *Hawsepipe*

- Measure and record chain wire diameter at the bottom of the hawsepipe.

- Check and report rubbing casting wear. If the rubbing casting is missing, then check for rusting and wear of the bell mouth.

- Ensure that chain is securely attached to rubbing casting.

### e. *Bottom Jewelry*

- Identify and report each type of chain component between lower tension bar eye and riser chain.

- Measure and record component length and wire diameter.

- Report any observed wear or corrosion.

### f. *Cathodic Protection System on Buoy*

- Record number, size, and location of installed anodes.

- Ensure that each anode is securely attached to buoy.

- Use an underwater voltmeter or portable voltmeter and silver/silver chloride reference electrode as described in Section 6.4, measure the potential of the bottom of the buoy in at least three locations and record the potentials. The guidelines for interpretation of the potentials given in Table 7-1 also apply to steel buoy bottoms.

Table 7-1. Underwater Voltmeter Values for Steel Structures.

| Voltage Measured (V) | Description |
|---|---|
| 0.0 to –0.7 | Steel is cathodically unprotected. The rate of corrosion depends on the effectiveness of paint or tar coatings, marine growth, and local water chemistry and water currents. On some structures, the hard layer of marine growth may provide some protection. The closer to 0 volts the more active the corrosion potential. Note: -0.6 volt is the potential of bare, unprotected steel in seawater. |
| -0.7 to –0.82 | The steel is partially protected. |
| -0.83 to –1.1 | The steel is adequately protected. Cathodic protection systems are working effectively. |
| -1.1 or higher negative values | The steel is "overprotected." Note: Under some circumstances, the metal surface can be made more brittle when overprotected. Surface coatings may be damaged or "lifted off" by the excess formation of hydrogen bubbles. |

### 3. *RISER CHAIN SUBASSEMBLY*

#### a. *Links*

- Record chain type (cast, forged, Dilok).
- Using appropriate tools, clean the following components for measurements:

  (1) First three links below bottom jewelry.
  (2) Three links just above ground ring.
  (3) Three links about halfway in between these two areas.
  (4) If the riser contains more than one shot of chain, clean links and take measurements at both ends and near the center of each shot.

- In a nonriser-type mooring clean:

  (1) First three links of each leg just below buoy's padeyes.
  (2) Three links just above mudline.
  (3) Three links about halfway in-between.

- Take and record double-link measurements of cleaned links.
- Record length of one of the links cleaned at each area.
- Check for and record manufacturer's markings.
- Check for pitting, measure diameter and depth of any pits found, and record results.
- Record water depth below buoy where each measurement is taken.

#### b. *Connecting Hardware*

- If visible, identify and record type of each (shackle, detachable link, anchor joining, etc.). Detachable links should be found on either side of the shackle and at the top and bottom of each chain shot.

- Record component's overall length and wire diameter.

- Report any loose, broken, or missing parts. If visible, note condition of tapered locking pin in a detachable link.

- Record water depth below buoy of each connecting component.

- Record any manufacturer's markings.

Past experience has indicated that the most severe wear occurs at the shackle connecting the mooring buoy padeye with the top link of chain. Special attention is required to inspect this shackle:

- Measure the least diameter of the shackle pin.

- Inspect whether the pin exhibits any outward movement.

- Check and record the condition of locking wire or pin at the end of the shackle pin.

*c. Swivel*

- Each riser subassembly should contain a swivel. Record swivel depth.

- Check swivel for marine growth.

- Record any manufacturer's markings.

### *4. GROUND RING SUBASSEMBLY*

- Three typical types of ground ring assemblies are:

  (1) A ground ring with four pear links attached.
  (2) A ground ring with four anchor joining links.
  (3) A ground ring with four shackles.

- Record type of ground ring assembly observed.

- Measure and record inside diameter (ID) of ring.

- Check and report any distortion of ring from circular that would indicate overstressing.

- Record height of assembly above bottom, or if the water is too deep, record depth below buoy.

- Using calipers, measure wire diameters of links attached to the ground ring and record results.

- Record any manufacturer's markings.

### 5. *GROUND LEG SUBASSEMBLY*

#### a. *Links*

- Record chain type installed (cast, forged, Dilok).

- Using appropriate tools, clean the following for measurements:

  (1) First three links of each leg below the ground ring.
  (2) Three links above mudline.
  (3) Three links about halfway in-between these two areas.

- Measure and record double-link measurements of the cleaned links. If one or more legs should extend considerable distances before entering the bottom, clean links and take measurements at both ends and near the center of each visible shot. If chain is not in tension, single-link measurements should be taken and recorded.

- Record length of one of the links at each area.

- Check for and record manufacturer's markings.

- Check for pitting, measure diameter and depth of any pits found, and record results.

- Record each anchor leg length from ground ring to bottom and from where it touches bottom to the point it becomes buried.

- Using a compass, note and record the relative bearing of each leg from the ground ring.

- Repeat above steps for three links at each end of tandem anchor connecting chain (if visible).

#### b. *Connecting Hardware*

- Identify and record component type (shackle, detachable link, anchor joining, etc.).

- Record component's overall length and wire diameter.

- Report any loose, broken, or missing parts.

- Record any manufacturer's markings.

- Record position of each connecting component by leg number and number of feet from ground ring.

***c. Swivel***

- Each anchor leg subassembly may contain a swivel. If located, record position by its leg number.

- Record any manufacturer's markings.

***6. ANCHOR ASSEMBLY (if visible)***

***a. Anchor***

- Identify and record type. Note whether or not anchor has stabilizers.

- Attach a pop float and record its bearing from the buoy using a compass.

- Determine and record anchor's orientation (i.e., flukes buried, flukes up, anchor on its side, anchor facing the wrong direction, etc.).

***b. Repeat all of the above for the tandem anchor.***

***c. Connecting Hardware***

- Identify and record component type and location.

- Record component's overall length and wire diameter.

- Report loose, broken, or missing components.

***7. PROPELLANT-EMBEDMENT ANCHORS (if visible)***

***a. Swage Fittings***

- Check for any loose, broken, or missing pins or parts.

- Check for fraying of the wire rope pendant where it enters the swage fitting and report any noted.

***b. Pendant/Downhaul Cable***

- Measure and record the wire diameter.

- Check for fraying, kinking, "birdcag-ing," or rusting of the cable and report any noted. Look for a "necking down" of the wire that may indicate the existence of a corrosion cell.

- Record the amount (in feet) of wire pendant visible between the anchor leg and the point that the pendant enters the bottom.

- Report any evidence of pendant cable movement on the bottom.

- Inspect and record any sign of wire rope pullout at the terminations.

### *8. EQUALIZER (SPIDER)*

- Check for rust and wear.

- Note the amount of marine growth located within the equalizer.

### *9. CHAIN CATHODIC PROTECTION SYSTEM*

#### *a. Anodes*

- Record anode size and location on the chain.

- Observe and record anode condition and determine whether or not its consumption is uniform.

- Record the color and estimate the thickness of the oxidation coating.

- Ensure secure attachment to the chain and the continuity wire.

- Using an underwater voltmeter, measure and record the chain's electrical potential.

#### *b. Electrical Continuity-Cables/Clips*

- Check and record cable's secure connection to the chain.

- Probe chain every 15 feet until anchor is reached or chain disappears into the bottom and record the potentials.

- The risers of mooring buoys are commonly protected from corrosion using sacrificial anodes. These anodes are consumed by galvanic action while providing protection for the chain. In order to be effective, electrical continuity between the anode and all chain links must be provided. For the ground legs that are often slack, a connecting wire attached to the

chain provides this continuity. The tension in riser chain is normally sufficient to maintain electrical continuity.

**7.4.2 Cathodic Protection Inspection.** The most common method of cathodically protecting compliant moorings involves the use of 150- to 500-pound zinc anodes attached to buoy hulls and each shot of chain. Underwater voltmeters are used to determine the effectiveness of the system by measuring relative electrical potentials of the buoy and chain at certain distances from the anode. The electrical potential of the metal is the charge of the metal compared to a standard reference electrode, typically silver/silver chloride. Steel that is adequately protected from corrosion should have electrical potentials that fall between -0.80 and -0.90 volts (Table7-1).

A portable voltmeter and portable reference electrode can be used to measure the potential on buoy hulls, but due to water depth and questionable electrical continuity, an underwater voltmeter is required for inspection of the cathodic protection system on chain.

A greater potential indicates that the anode is overworking and serious damage could occur to the metal, while a lesser potential indicates that the system is not operating effectively and corrosion may be occurring.

Compliant mooring inspection divers will normally use a self-contained voltmeter, which consists of a digital display, surface readout facility, and rechargeable battery. Underwater voltmeter readings must be taken at 20-foot intervals on the chain, on each side of each anode, at each end at the continuity cable, and on each side of each swivel. Whenever readings are taken, potentials, depth, and element measured should be recorded. Note that a moored vessel can affect the cathodic protection system on the mooring buoy and chain and cause the readings to be either higher or lower than normal.

## 7.5 Equipment and Tools Required

Chain measurements are best made with precut "go-no go" gauges, calibrated at 90 and 80 percent of original wire diameter. Calipers (24-inch minimum) are also required, with the measurements taken off a ruler attached to a Plexiglas slate. A 100-foot tape and scales 1, 2, and 3 feet long with large numbers suitable for photo documentation will be required. A diver's compass and accurate depth gauge, as well as survey buoys, will also be required.

The effectiveness of cathodic protection systems is measured as follows:

- On mooring buoys - either an underwater voltmeter or a portable voltmeter and portable reference electrode can be used.

- On chain - an underwater voltmeter is used.

To record any findings underwater, a grease pencil and Plexiglas slate are required. An underwater camera is required for photographic documentation, and a video recording system may be required. An inclinometer is required for obtaining the angles of mooring chains in nonriser-type moorings and spread moorings. Marker tags are used to relocate or mark links or accessories. Transits and targets are required for locating buoy positions. Because divers need high mobility, and because of the depth of water in which they will be working, the cleaning operations to be performed for inspection work generally require only hand tools, such as wire brushes and scrapers.

At times ROVs may be used to supplement mooring inspections.

# 8.0 CONCRETE STRUCTURES

## 8.1 Types of Marine Concrete Structures

Concrete is widely used in the marine environment as a construction material because of its many desirable properties. In its plastic state, concrete is easily mixed, handled, transported, and placed into forms. The strength of concrete can be regulated by adjusting the quantities of cement, aggregate, water, admixtures, and, in particular, the water-to-cement ratio. This ratio is one of the prime considerations in concrete mix design not only to provide adequate concrete strength, but, equally important, to provide long term durability of concrete in the harsh marine environment.

The performance of a concrete structure is most affected by the care taken in its construction and installation. Properly made concrete is highly durable in marine applications, exhibiting resistance to corrosion of reinforcing steel, chemical deterioration, weathering, erosion, and structural damage. Concrete is relatively strong under compressive loading, and with steel reinforcing resists bending and tensile forces. Concrete can be cast in place at the job site, precast into the required shape at a concrete plant and shipped to the site, or prestressed before installation to accept additional loading. With proper procedures, concrete can be rapidly placed underwater where it will harden into good quality concrete.

Circular or square concrete piles (Figure 4-2) are widely used to support piers, wharves, and other structures. Concrete is used as a decking material for many waterfront facilities and in retaining wall structures, such as those needed for closed piers and wharves, bulkheads, quaywalls, dry-docks, and seawalls. It is also used in pavements, bridge foundations, boat loadings and ramps, breakwaters, undersea cable and pipeline stabilization, and offshore structures.

## 8.2 Deterioration of Marine Concrete

The most common damage resulting from the premature deterioration of concrete structures in or near seawater is cracking and loss of material (or cross section). Softening of the concrete due to chemical action is another form of damage but less common than cracking. As shown in Table 8-1, the damage to concrete is generally most severe in the splash and tidal zones, but does occur in all zones. The different exposure zones are shown in Figure 5-1.

Deterioration of concrete waterfront structures is caused primarily by:

- Corrosion of steel reinforcement
- Repetitive freezing and thawing of moist concrete
- Abrasion
- Chemical deterioration from saltwater
- Structural overloading
- Shrinkage
- Swelling

Concrete damage is found by walking the pier deck, by inspecting below the pier deck with a small boat or barge, and by underwater inspections. The primary method of inspecting concrete is by visual observation. Most durability problems will be detected visually using hand

tools such as pick and hammer. Only after problems are detected should other inspection methods such as probes, coring or sonic test equipment be considered.

Table 8-1. Types of Damage in Marine Concrete.

| Description of Damage | Zone (Location)[a] | | Common Causes of Damage | | | | | |
|---|---|---|---|---|---|---|---|---|
| | Observed | Most Severe | Corrosion of Reinforcement | Freeze-Thaw | Abrasion | Sulfate Attack | Chemical Reaction of Aggregates | Structural Overload |
| Cracking | All | T | X[b] | X | | | X | X |
| Loss of material-[c] Exposed Reinforcement and/or Aggregate | All | S, T | X | X | X | X | X | X |
| Material | S, T, Su | S, T | | | | X | | |

[a]A = Atmospheric zone: S = Splash zone; T = Tidal zone; Su = Submerged zone; M = Mud zone (see Figure 3-11).
[b]Rust stains on the concrete surface are generally a symptom of corrosion of the reinforcement.
[c]Loss of material from spalling, scaling, disintegration.

The three most common visual signs of concrete deterioration in marine structures are: cracking, disintegration, and spalling. Disintegration is defined as an overall decay of the concrete involving loss of strength of the cement and sand paste and subsequent loosening or loss of coarse aggregate particles. Spalling is defined as a localized area or fragment of concrete falling away from the structure. Both disintegration and spalling can expose reinforcing steel.

The causes for each symptom of deterioration are many and varied, and in most cases of progressing deterioration, they occur simultaneously. Much concrete deterioration in the marine environment starts as a result of poor construction techniques and inadequate inspection and quality control during construction. To develop a suitable and adequate concrete repair procedure, the cause of deterioration must be determined. Causes of concrete deterioration are described.

**8.2.1 Corrosion of Reinforcing Steel.** With the exception of mass gravity structures, most marine concrete structures use steel reinforcement. This reinforcement, to be most effective, is nearly always located within a few inches of the concrete surface, making the steel susceptible to corrosion if it does not have adequate cover of good quality concrete. Corrosion is more likely to occur if the concrete is overly porous or if cracking is initiated by some action.

The reinforcing steel corrosion products (rust) can increase the volume of the rusted area up to eight times. This leads to cracking of the concrete cover in lines parallel to the reinforcing steel. Eventually spalling results, and in cases of close reinforcement spacing, a complete delamination of the concrete surface can occur.

All concrete is porous to some extent. The degree of porosity is dependent primarily on the water/cement (w/c) ratio of the concrete mix and on good construction practices. The lower the w/c ratio, the more dense (less porous) the concrete which limits the rate at which water, dissolved oxygen, and chloride ions reach the reinforcing steel, and lengthens the time for corrosion of the rebar to damage the concrete.

For example, reinforced marine concrete made with a w/c ratio of 0.6 to 0.7 (7 or 8 gallons of water per 94-pound sack of cement) will show rebar corrosion, cracking, and spalling in a few years, whereas well made concrete with a w/c of 0.4 (about 4-1/2 gallons of water per sack of cement) will likely serve several decades or more before serious deterioration occurs.

**8.2.2 Freeze/Thaw Deterioration.** Freeze/thaw deterioration is the freezing of absorbed moisture or water in porous concrete exposed to subfreezing temperatures. This is one of the most common causes of concrete deterioration in the tidal and splash zones. Upon freezing, this entrapped water expands and cracks the concrete. Upon thawing, the cracked surface disintegrates. Repeated cycles of freezing and thawing can lead to partial or even total loss of the concrete cross section, thus exposing the reinforcing steel which then rapidly corrodes as illustrated in Figure 8-1. The best prevention of freeze/thaw damage is to use air-entrained concrete with a rich cement content and a low water/cement ratio.

Precast concrete piles may have a cast-in-place jet pipe that was not filled with concrete after the pile was driven. When the water in the pipe freezes, it can cause longitudinal cracks in the pile, as illustrated in Figure 8-2.

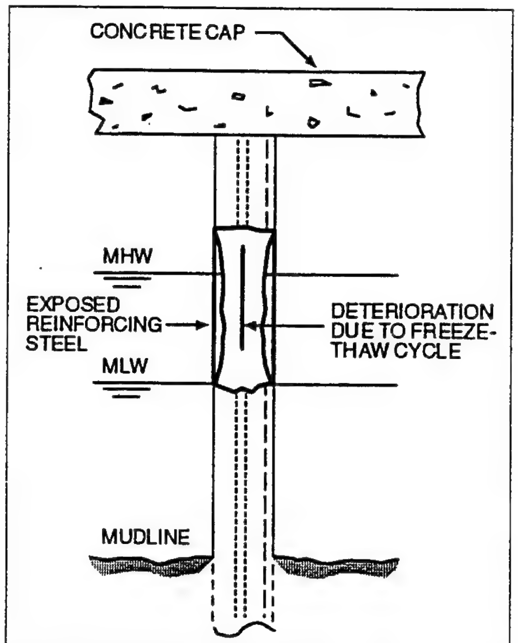



Figure 8-1. Loss of concrete cross section due to freeze/thaw cycle.

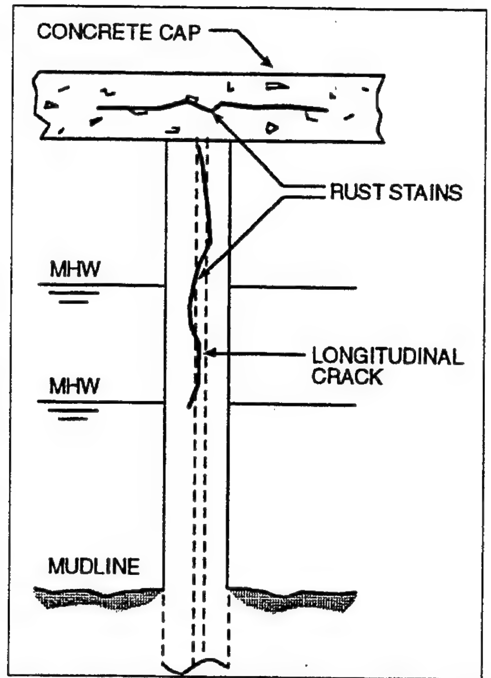


Figure 8-2. Longitudinal cracks in precast concrete pile due to freezing of cast-in-place jet pipe.

**8.2.3 Abrasion Wear.** Abrasion is defined as erosion of a concrete surface by the physical action (impact and rubbing) of external loadings or abrading agents. Deck slabs are subject to abrasion by vehicular traffic and loading equipment. Deck edges and wharf faces at berthing spaces without adequate fendering are abraded by moored vessels. Frequently, concrete piles and walls are abraded in the tidal zone by floating debris moved by currents, waves, propeller wash, and tide changes. Less frequently, submerged concrete, especially at the mudline, is abraded by silt, sand, and debris churned up by moving water. Figure 8-3 illustrates the effects of abrasion on a concrete pile.

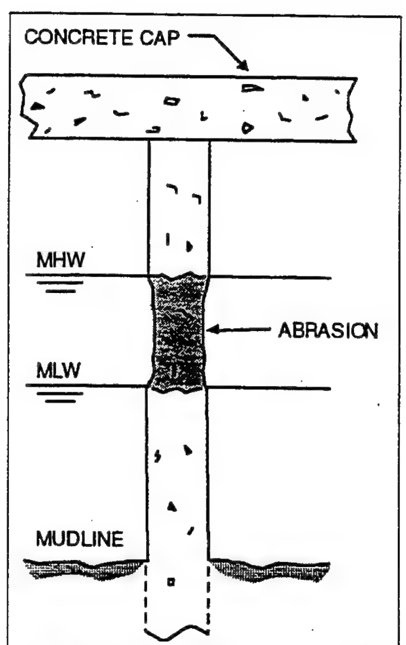


Figure 8-3. Abrasion effects on a concrete pile.

**8.2.4 Chemical Deterioration.** The most significant and serious saltwater chemical reaction to hardened concrete is the combining of sulfates in seawater with chemicals in the cement paste, referred to as sulphate attack. This reaction can produce internal expansion and cause cracking. More commonly, however, the hydrate cement paste looses strength and becomes soft. Aggregate particles become exposed or fall from the concrete mass because of the weak cement paste.

**8.2.5 Axial Overloading.** Deterioration of concrete piles from axial overloading can be a cause of eventual failure of the pile. Overloading can result from superimposed "dead" and "live" loads exceeding the bearing capacity of the pile, and also from overstressing at the time of pile driving. Pile driving overloading often results in hairline cracks at the top of the pile or circumferential cracks at other locations along the pile that are difficult to see, as illustrated in Figure 8-4. As marine growth covers the pile, the cracks become extremely difficult to detect.

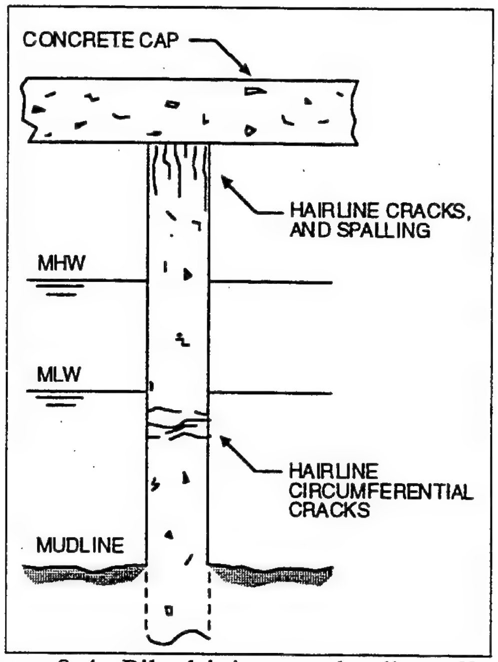


Figure 8-4. Pile driving overloading effects.

**8.2.6 Shrinkage.** Shrinkage or contraction can occur from moisture or temperature changes. Hardened concrete that looses internal water due to evaporation will shrink. Any temperature decrease of the concrete will cause contraction. The major cause of microcracks within concrete is from high temperatures generated from the normal hydration of cement. The concrete hardens at a high temperature and later cools to ambient temperatures. Precast concrete members that have been steam cured are particularly susceptible to microcrack formation. If the shrinkage or contraction is restrained, internal stresses may develop in sufficient magnitude to cause significant cracks in the structure.

Variations in atmospheric temperature cause a change in temperature of a hardened concrete mass, which results in volumetric changes. Provisions must be made to permit this expansion and contraction process to take place. Failure to do so will result in contraction stresses (tension), which may cause cracking, or expansion stresses (compression), which may lead to spalling.

**8.2.7 Swelling.** Concrete that increases in moisture content by absorbing water or increases in temperature will swell or expand. Typically, swelling by water absorption is not a concern unless precast dry-concrete members are used. Temperature increases from daily and seasonal changes may cause cracking in some concrete members.

**8.2.8 Other Deterioration Factors.** The proceeding has discussed deterioration in concrete caused by improper selection or proportioning of concrete materials, faulty construction methods and procedures, and attack by environmental forces. Of equal importance, and a major cause of much concrete deterioration, is poor design of concrete structural details.

A few examples of poor design and construction details that contribute to concrete failure and deterioration are:

- Congestion of reinforcing steel
- Lack of adequate cover for reinforcing steel
- Abrupt change in size of section
- Reentrant corners
- Lack of chamfers and fillets at corners
- Rigid joints between precast units
- Construction joint leakage
- Poorly designed scuppers, drips, and curb slots
- Inadequate drainage
- Too little gap at expansion joints
- Incompatibility of materials or sections

## 8.3 Concrete Inspection Procedure

**8.3.1 Surface Inspections.** The areas below should be inspected to ensure a thorough inspection of concrete structures and their attachments above water. Include annual load testing of the pier decking if heavy equipment or vehicles are to be driven onto the pier. Areas where the inspector should be particularly watchful for signs of deterioration include:

(1) Inside corners and areas where radical changes occur in size of deck sections.

(2) Deck expansion joints where insufficient gap is allowed, rigid joints between precast units, and construction joints in general.

(3) Poorly designed scuppers, drips, and curb slots, and other areas where inadequate drainage exists.

(4) Joints between the deck and pile cap, expansion joints where insufficient gap is allowed, and rigid joints between precast piles and cast-in-place pile caps.

The inspector should be alert for any change in appearance of the concrete surface and any change in sound from the hammer. Chemical attack will be indicated by erosion of surface material or by cracking on the surface. Freeze-thaw deterioration will appear as erosion of surface material. A hammer or gad (sharp pointed tool), should be used to chip or probe the surface to detect the depth of deterioration.

Corrosion of reinforcement materials can be detected from rust stains on the surface. More advanced stages of corrosion are indicated by cracks that run parallel to the steel reinforcing bars. At times, corrosion is hidden from view, but will be indicated by a hollow sound from the hammer. This can occur on heavily reinforced slabs, such as pier decks, where the reinforcement has corroded enough to spall a layer of concrete at the level of the reinforcing mat.

Cracks found on the surface of a concrete structure should be given careful attention. Sketches should be made to show the length and direction of the cracks. Overall cracking patterns and changes in crack length, width and direction with time are meaningful data to a structural engineer. Photographs are helpful, but only as a supplement to the sketches.

If there is evidence of significant deterioration, more detailed NDT techniques may be employed in a scheduled Level III inspection. Refer to the Level III Test Procedures for Concrete Inspection for mechanical and electrical test methods. The plan and sampling techniques shall be tailored to the specific areas of concern.

### 8.3.2 Underwater Inspection

**8.3.2.1 Visual Inspection.** Levels I and II visual inspection of concrete waterfront structures should proceed as shown in Table 8-2.

Table 8-2. Concrete Structure Underwater Inspection Checklist.

| Checkpoint | Description |
|---|---|
| 1 | Inspect the structure beginning in the splash/tidal zone. This is where most mechanical and biological damage is normally found. |
| 2 | Clear a section about 18 to 24 inches in length of all marine growth. |
| 3 | Visually inspect this area for cracks, abraided surface spalling, or mechanical damage, and exposed reinforcing steel. |
| 4 | Sound the cleaned area with a hammer to detect any loose layers of concrete hollow spots in the pile, structure, or soft concrete. A sharp ringing noise indicates sound concrete. A soft surface will be detected, not only by a sound change, but also by a change in the rebound, or feel, of the hammer. A thud or hollow sound indicates a delaminated layer of concrete, most likely from corrosion of steel reinforcement. |
| 5 | Descend, visually inspecting the pile or structure where marine growth is minimal, and sound with a hammer. |
| 6 | Inspect in greater detail the base of mass structures, such as foundations, quaywalls, breakwaters, or bridge piers. These types of structures are prone to undermining by wave and current action, which, if not rectified, could lead to failure of the structure. |
| 7 | At the bottom, record the water depth along with any observations of damage on a Plexiglas slated. |
| 8 | After returning to the surface, immediately record all information into the inspection log. NOTE: If signs of deteriorations are found, then a Level III inspection, involving either nondestructive or destructive tests, may be required. Refer to the Level III Test Procedures for Concrete Inspection for mechanical and electrical test methods. |
| Exposed Area Under Pier or Along Wharf or Dolphin Assembly | |
| 9 | Check pile caps and bearing, batter, and fender piles for damaged or broken members, cracks, and spalling of concrete, rust stains, and exposed reinforcing steel. |
| 10 | Sound the piling or structure with a hammer to detect any loose layers of concrete or hollow spots. A sharp ringing noise indicates sound concrete. A soft surface will be detected, not only by a sound change, but also by a change in the rebound, or feel, of the hammer. A thud or hollow sound indicates a delaminated layer of concrete, most likely from corrosion of steel reinforcement. **NOTE**: If signs of deterioration are found, then a Level III inspection, involving either nondestructive or destructive tests, may be required. Refer to the Level III Test Procedures for Concrete Inspection for mechanical and electrical test methods. |

**8.3.2.2 Level III Nondestructive Inspection of Concrete.** The qualitative data obtained from visual inspections are sometimes inadequate to accurately assess the condition of the structure. In these instances, quantitative data obtained from nondestructive testing instruments can assist the facilities engineer in determining the condition of the structure. Three specialized instruments have been developed for underwater inspection of concrete structures. These instruments are the:

- **Magnetic rebar locator** - used to determine the location and orientation of rebar in concrete structures and to measure the amount of concrete cover over the rebar.

- **Rebound hammer** - used to evaluate the surface hardness of the concrete and obtain a general condition assessment.

- **Ultrasonic system** - used to obtain a general condition rating and indication of overall strength of the concrete based on sound velocity measurements through a large volume of the structural element.

Each instrument consists of an underwater sensor connected to a topside deck unit through an umbilical cable. The deck unit contains the signal conditioning electronics and data acquisition system. To operate the instruments, the diver has to position the underwater sensor on a previously cleaned portion of the structure surface and a person topside must operate the data acquisition system in order to collect and store the data. Each instrument is independently operated and provides unique information to help assess the condition of the concrete structure.

## 8.4 Equipment and Tools Required

To perform a thorough inspection, the marine growth on the structure must be removed. A "Barnacle Buster" or pneumatic chipping gun is an efficient method of removing marine growth from concrete surfaces. Various types of high-pressure water jet cleaning systems are also effective. Exercise care in the use of these methods because they may further damage a deteriorating concrete structure. If minimal marine growth is found in the splash/tidal area, small hand tools, such as wire brushes and scrapers, are sufficient. A hammer for sounding and an accurate water-depth gauge will be required. Record observations on a Plexiglas slate with a grease pencil. Use underwater video cameras for permanent visual documentation.

**8.4.1 Magnetic Rebar Locator**. The magnetic rebar locator is an instrument that detects the disturbances in a magnetic flux field caused by the presence of magnetic material. The magnitude of this disturbance is used to determine the location and orientation of rebar in concrete structures and to measure the amount of concrete cover over the rebar. The system consists of an underwater test probe, an umbilical cable, and a topside data acquisition unit (DAU) including printer.

The test probe consists of two coils mounted on a U-shaped magnetic core. A magnetic field is produced in one coil and the disturbance-induced magnetic field in the rebar is measured in the other coil. The magnitude of the induced current is affected by both the diameter of the rebar and its distance from the coils. Therefore, if either of the parameters is known, the other can be determined. By scanning with the probe until a peak reading is obtained, the location of the rebar can also be determined. A maximum deflection of the meter needle will occur when the axis of the probe poles are parallel to and directly over the axis of a reinforcing bar, thus indicating orientation.

The underwater rebar locator is calibrated for rebar that varies from No. 3 to No. 16 in size. The meter can be used to measure the depth of concrete cover over rebar in the range of 1/4 to 8 inches thick, or conversely, it can measure the diameter of the rebar. The best accuracy (±10 percent) is obtained for concrete cover less than 4 inches thick.

- **System Limitations**. The presence of other metallic objects in the vicinity where the measurements are being made can affect the operation of the rebar locator. For example, in heavily reinforced structures, the effect of nearby rebar cannot be eliminated and accurate depth readings are difficult or impossible. If the separation of two parallel rebars is at least three times the thickness of the concrete cover, this effect can be neglected. The presence of rebar perpendicular to the axis of the underwater probe has less effect on the measurement of concrete cover than that of parallel rebar, and in most instances it can be ignored.

**8.4.2 Rebound Hammer**. The underwater rebound hammer system, is a surface hardness tester that can be used to obtain a general condition assessment of concrete. The system consists of an underwater rebound hammer, an umbilical cable, and a topside data acquisition unit (DAU) including printer. The rebound hammer is mounted in a waterproof housing which contains an electrical pickup to sense the position of the rebound mechanism. The umbilical connects the underwater rebound hammer to the DAU that contains the signal conditioning electronics and data acquisition system.

The rebound hammer correlates the rebound height of a spring-driven mass after it impacts the surface of the concrete with the compressive strength of the concrete under test. The spring-driven mass slides on a guide rod within the tubular housing. When the impact plunger is pressed firmly against the concrete surface, a trigger releases the spring-loaded mass causing it to impact the plunger and transfers the energy to the concrete surface. The mass then rebounds and the rebound height is correlated to the surface hardness of the concrete.

A general calibration chart relates the rebound number to cube compressive strength for the underwater rebound hammer. The pressure housing has a depth rating of 190 feet and it is pressure compensated at 5 psi over the ambient pressure. Air is supplied to the rebound hammer from a scuba tank through the umbilical cable via an external pressure regulator to maintain the positive pressure differential inside the housing.

- **System Limitations**. The following characteristics of concrete can affect the correlation of the rebound number with the actual surface hardness and should be understood before using the instrument:

(1) Higher rebound numbers are generally obtained from smoother surfaces and the scatter in the data tends to be less. Minimizing the data scatter increases the confidence in the test results. Therefore, underwater concrete surfaces must be thoroughly cleaned and smoothed with a carborundum stone (or similar abrasive) before measurements are taken.

(2) Water-saturated concrete tends to show rebound readings approximately 5 points lower than for the same concrete tested dry. This affects the comparison of data taken above and below the waterline.

(3) Type of aggregate and cement affects the correlation of the rebound numbers with actual compressive strength of the concrete under test. A calibration curve is required for each particular concrete mix to assure accuracy. Since this is not practical for most situations, the data should only be used for making comparative measurements from one location to another within a uniform concrete structure.

Because of these limitations, the estimation of concrete compressive strength obtained with a rebound hammer is only accurate to about ±25 percent. This applies to concrete specimens cast, cured, and tested under the identical conditions as those from which the calibration curves were established. The rebound hammer is primarily useful for checking surface compressive strength or surface hardness and uniformity of concrete within a structure. It can also be used to compare one concrete structure against another if they are known to be reasonably similar.

**8.4.3 Ultrasonic System**. The ultrasonic system is used to obtain a general condition rating and indication of overall strength of the concrete based on sound velocity measurements

through a large volume of the structural element. It is recommended that the underwater ultrasonic system be used primarily for checking the uniformity of concrete from one test location to another in a given structure. If the data consistently indicate poor or very poor quality concrete, core samples must be taken and standard compression tests performed to confirm the results.

The system consists of two different underwater transducer holders for direct and indirect sound velocity measurements. An umbilical cable connects either the direct or indirect transducer holder to the topside DAU. The DAU contains most of the signal conditioning electronics and data acquisition system.

Ultrasonic techniques use the transit time of high-frequency sound waves through concrete to assess its condition. Ultrasonic testing procedures for concrete have been standardized by ASTM Standard C597 and test equipment is available from commercial sources for in-air testing. Measuring sound velocity in concrete requires using a separate transmit and receive transducer to avoid energy scattering and reflection problems. Sound velocity is calculated by measuring the time required to transmit over a known path length. The average sound velocity obtained should only be used as an indicator of concrete quality and not as a measurement of compressive strength. Table 8-3 presents some suggested condition ratings for concrete based on sound velocity measurements.

Table 8-3. General Condition Rating Based on Sound Velocity.

| Condition Rating | Sound Velocity (ft/sec) |
|---|---|
| Excellent | >15,000 |
| Good | 12,000 – 15,000 |
| Questionable | 10,000 – 12,000 |
| Poor | 7,000 – 10,000 |
| Very Poor | <7,000 |

The two methods used to measure sound velocity in concrete are direct and indirect. The most preferred method is direct transmission where the transducers are positioned on opposite sides of the test specimen and the waves propagate directly toward the receiver. This method provides maximum sensitivity with a well-defined path length.

Indirect transmission is used when only one surface of the concrete is accessible, such as a concrete retaining wall: both transducers are placed on the same side of the concrete. With this method, energy scattered by discontinuities within the concrete is detected by the receive transducer.

- **Transducer Holders**. Two types of transducer holders are provided with the ultrasonic system. The direct transducer holder is used to examine structures with accessible opposing surfaces; for example, concrete piles. The indirect transducer holder is used to examine structures with only one accessible surface; for example, concrete bulkheads.

The direct transducer holder framework can be adjusted to accommodate concrete pile sections that range from 8 inches to 24 inches thick. The digital display of sound wave transit time provides feedback to help the diver position the transducer holder for optimum results.

The indirect transducer holder is very similar to the direct transducer holder in operation except for the path length measurement that is fixed at 12 inches. A suction cup was added to the

indirect holder to force the transmit transducer firmly against the concrete surface under test and provide a reaction force for the diver. A small suction pump is used to pump water from the cup to provide a holding force of about 25 pounds depending on the surface condition of the concrete.

- **System Limitations**. Results obtained with the ultrasonic test system are affected by the following factors which influence the quality of the data:

(1) **Concrete Surface Finish** - The smoothness of the surface under test is important for maintaining good acoustical coupling between the transmit transducer and the surface of the concrete. A coupling agent, such as silicone grease, must be placed between the transmit transducer and the concrete surface to transfer maximum energy. If a coupling agent is not used, the transmitted signal will be severely attenuated which results in large errors in the measurement of the transit time.

(2) **Reinforcing Steel** - Sound velocity measurements taken near steel reinforcing bars may be higher because the sound velocity in steel is from 1.2 to 1.9 times the velocity in concrete. The effect is small when the axis of the rebar is perpendicular to the direction of sound propagation and the correction factors are on the order of 1 to 4 percent depending on the quality of the concrete. If the axes of the rebar are parallel to the direction of sound propagation, reliable corrections are difficult. Therefore, it is recommended that sound transmission paths be chosen that avoid the influence of the rebar.

(3) **Signal Detection Threshold** - The signal detection threshold of the ultrasonic system can cause erroneous transit time data to be recorded. This happens when the amplitude of the first peak of the received signal is below the threshold triggering level of the system. When the instrument detects a following peak, this causes an apparent transit time increase of one-half wavelength or more.

## 9.0 TIMBER STRUCTURES

### 9.1 Types of Timber Structures

Timber is used in marine applications as a construction material chiefly because of its low initial cost and the ease with which it can be procured, transported, and constructed into required shapes. Timber has a wide range of uses in marine construction. It can be used as pile material for waterfront structures such as piers and wharves; as decking and framing material for the upper portion of waterfront structures, fender piles, and dolphins; and as construction material for bulkheads and retaining walls. Softwood timber, such as fir and pine, must be pressure treated with an appropriate preservative before it is used in the marine environment. Hardwoods, such as oak and greenhart, which are often used in fender systems, are not treated.

### 9.2 Deterioration of Timber Structures

Timber structures are subject to deterioration from decay or rot, attack by marine borers and insects, splitting and checking brought about by drying shrinkage or by the alternate wetting and drying cycle within the splash zone, overloading, corrosion of connections, abrasion, and ice

heaving. Waterfront deterioration and damage is found by walking the pier, by inspecting dolphins and below pier decks in a small boat or barge, and by underwater inspections.

When inspecting above the water surface, the inspector should take maximum advantage of low tide conditions to visually observe the overall condition of the piling. This may lead to the determination that an underwater inspection is necessary. The underwater inspection should, on the other hand, take maximum advantage of high water conditions in order to compile the most comprehensive field data on existing conditions.

**9.2.1 Fungi and Rot Damage**. Several species of fungi exist by feeding on timber, causing a breakdown within the cellular structure of timber under attack. In the early stages, fungi attack is evident by a discoloration and softening of the wood accompanied by a fluffy or cottony appearance. Advanced attack will cause destruction of the wood cells and the appearance of fruiting bodies, such as mushrooms. Figure 9-1 illustrates the effect of timber rot. The rapidity of decay is dependent upon the fungi species, variety of wood, exposure, and climate. To live, fungi must have air, food (the wood), favorable temperature, and a moisture content of over 20 percent, which is generally higher than that of typical air-dried wood. Submerged timber will not rot because of a lack of air. Fungi growth takes place in all saltwater environments within the temperature range of 50 to 90°F. As the temperature level drops to freezing, fungi growth becomes dormant, but will reactivate when the temperature increases.

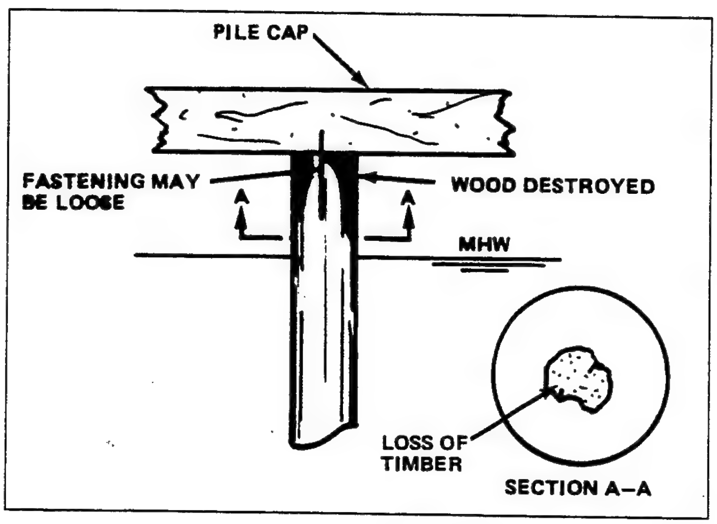



Figure 9-1. Effects of timber rot.

**9.2.2 Marine Borer and Insect Attack.** Marine borer attack is a very serious problem for timber structures in the splash and submerged zones. The ravaging effects of two large groups of marine invertebrates, the Teredo (commonly called shipworms) and the Limnoria (commonly called woodgribbles), are well-documented.

Shipworms are mollusks and are distantly related to the oyster and the clam, even though the adult form is worm-like in appearance. Shipworm species are found in nearly all saltwater harbors and oceans of the world, except for the colder waters of the Arctic and Antarctic regions.

Adult shipworms eject their young into the water at birth. These miniature animals are driven by tides and currents until they settle on firm surfaces or die. Should they settle on submerged timber during the first 48 hours of their life, they begin to change in physical appearance, with the body beginning to elongate, while two clam-like shells begin to auger into

the wood. The original hole in the timber surface created by the infant shipworm is no larger than the diameter of a pinhead.

As the shipworm continues to burrow and grow, it becomes more worm-like in appearance, with its body increasing in diameter to completely fill its burrow. In time, the animal orients its body parallel to the grains of the timber member. A calcium-like white excretion is left behind on the walls of the burrow track. As the exterior timber deteriorates via other means, these white trails become an obvious indication of shipworm presence. Eventually, the interior of a timber pile or beam under shipworm attack will become completely riddled with burrows, although externally no evidence of attack is apparent.

In tropical climates, shipworms have been known to grow to 6 feet in length and as much as 3 inches in diameter. In temperate climates, such as North America, shipworms generally range between 6 to 8 inches in length and about 1/2 inch in diameter. Figure 9-2 illustrates the damage caused by shipworms.

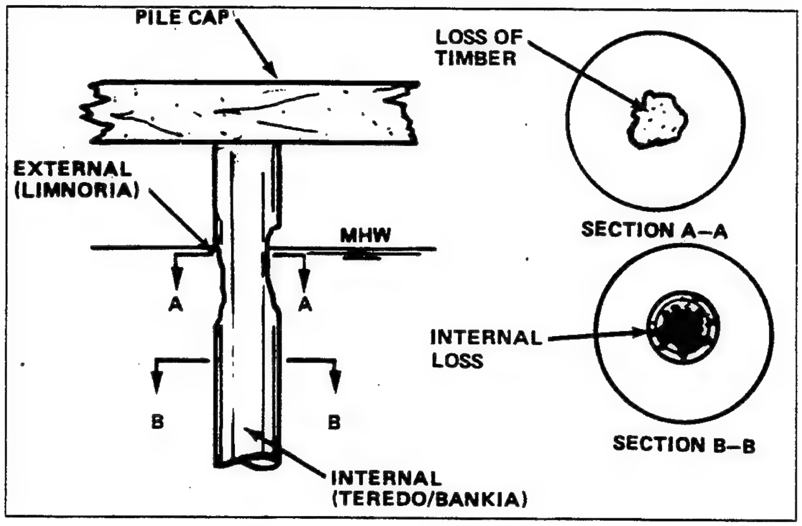



Figure 9-2. Damage to timber pile caused by shipworms.

Woodgribbles are crustaceans related distantly to the crab and shrimp family. They are quite small, averaging only 1/8 to 1/4 inch in length. This tiny organism is a voracious wood chewer, with its appendages and mouth developed for rasping and biting.

At birth, the woodgribble mother retains the young within a pouch until they develop sufficiently to fend for themselves. She then releases them within her furrow and they proceed to dig side furrows. Ordinarily woodgribbles do not burow deeply into the timber surface but limit their attack to shallow surface trenches. In timber piling, this results in a slow but continual reduction in pile diameter. Damage is most frequently found at the waterline or mudline, where the woodgribble population is the greatest. Severe attack will produce an hourglass appearance in piling, as illustrated in Figure 9-3, reducing the outside diameter of untreated pine or Douglas fir by up to 6 inches in 1 year.

Termites are the most destructive type of insect life to attack above-water and onshore timbers. They feed on the cellulose matter contained in timber. If termite damage is suspected, an ice pick makes an ideal tool to determine their presence, as a serious attack will eat away large interior portions of the wood just below the surface of the timber. Timbers most subject to attack are curbs and blocking on bulkhead fills.

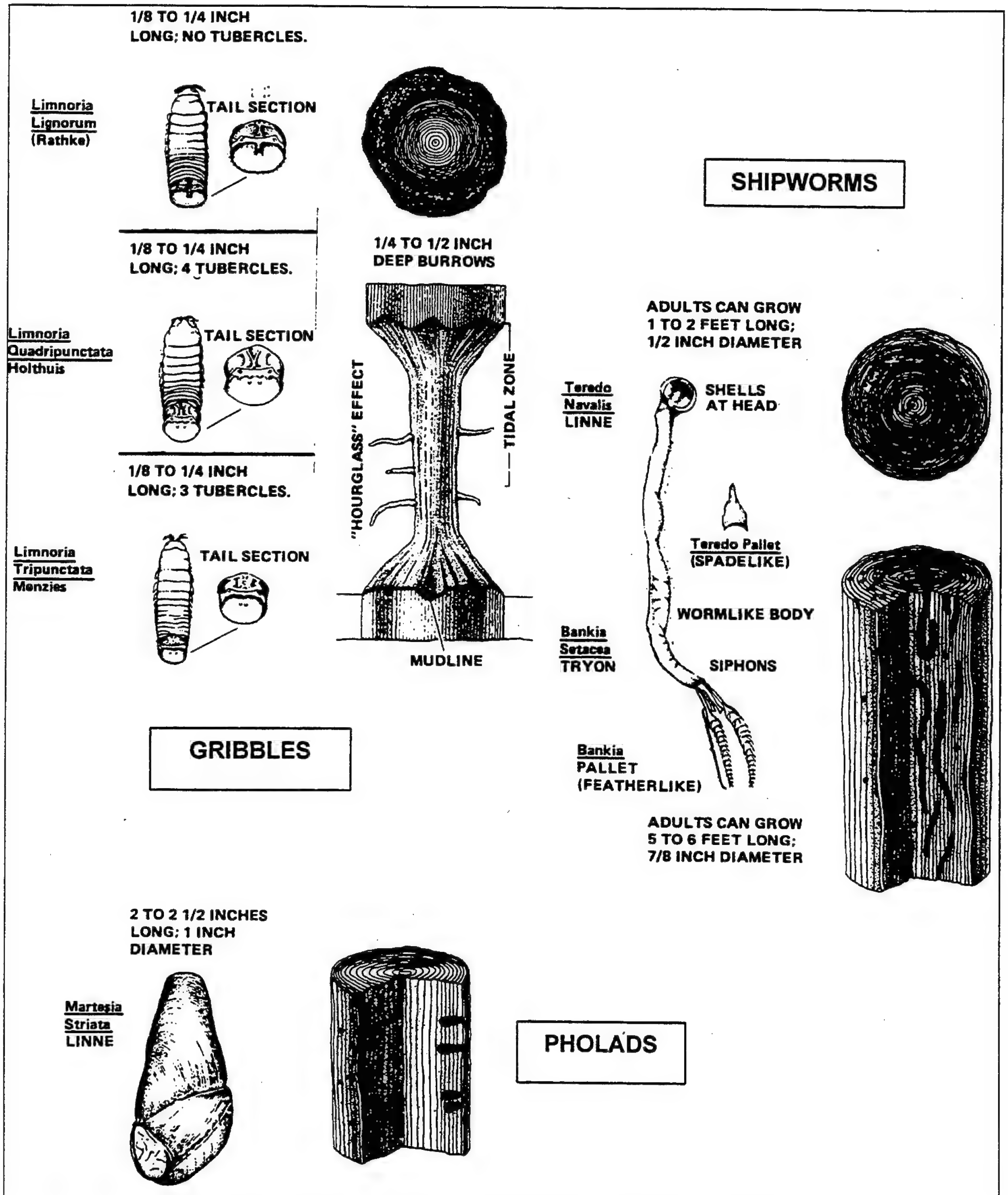



Figure 9-3. Marine Borers---Shipworms, Gribbles, and Pholads.

**9.2.3 Shrinkage Damage**. Drying causes timber to shrink. After installation this drying process continues, especially in hot dry climates, and the timber members split and check. This shrinkage also causes bolts to loosen in connections which, in turn, causes slippage and deflections in timber members and even distortion and weakening of the entire assembly. Ordinarily, splitting or checking is not serious and is allowed for in standard timber design specifications. However, splits and checks in excess of those allowed by the standard grading rules are potential troublemakers and should be closed.

Splitting and checking create an opening in the timber face or end that is an ideal means of access for insects and borers. These openings also tend to accumulate moisture and dirt, which can also easily lead to decay and rot. Should excessive moisture freeze, the split or check will widen.

**9.2.4 Overloading**. Axial and bending overloading of piles may be due to a continuous source of loading or to an infrequent type of loading. Material stored in a warehouse on a pier is a form of continuous loading. Short-time loading is exemplified by the impact of a vessel striking a pier or heavy vehicles passing over the deck of the pier. Timber fender faces are particularly subject to bending overloading during ship impact. Failure of one pile requires the adjacent piling to carry not only its own but also part of the damaged pile's load. Continual overloading will cause failure of the adjacent piling, leading to the eventual collapse of the entire structure. Figures 9-4 and 9-5 illustrate the effect of compression and bending overloading, respectively.

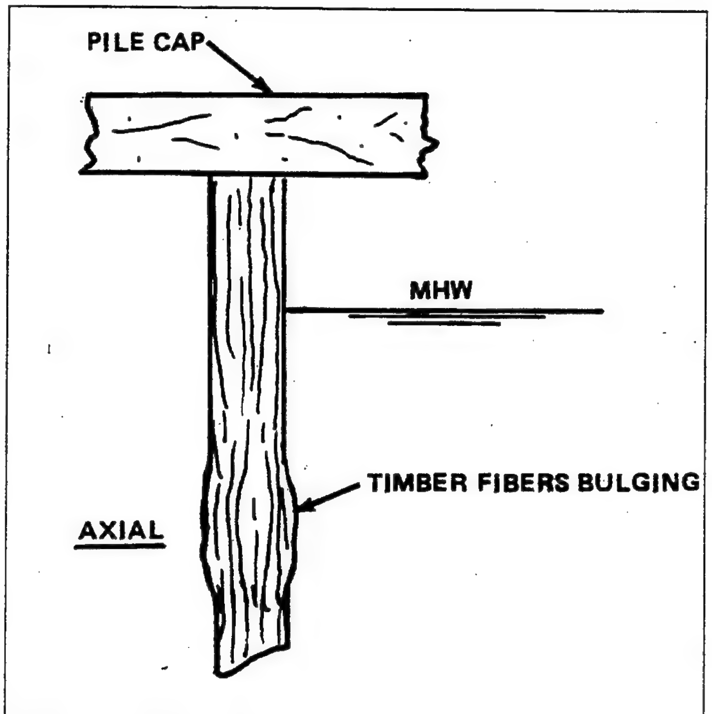



Figure 9-4. Compression overloading of timber pile.

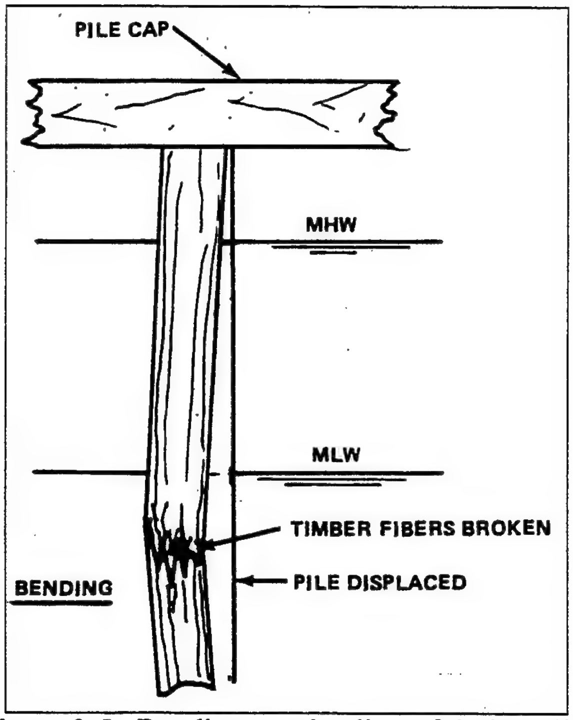


Figure 9-5. Bending overloading of timber pile.

**9.2.5 Structural Connection Corrosion**. The weak link in marine timber construction is the connecting hardware, since this steel hardware is subject to corrosion. Figure 9-6 illustrates one consequence of one type of hardware failure. Of prime importance to structure integrity are the proper sizing of pins and bolts and the use of Ogee washers in place of thin, flat plate steel washers. The pins and bolts are subject to corrosion, which is very difficult to prevent; therefore, all pins and bolts should be galvanized and oversized to provide a corrosion allowance, with a minimum diameter of 1-inch being supplied. More importantly, Ogee washers should be used. These tapered washers are made of corrosion-resistant wrought or cast iron and are equal in thickness to the bolt diameter. The outside diameter of Ogee washers is considerably greater than that of flat plate steel washers, making them far less subject to loosening under load because their greater bearing area prevents timber crushing.

Bolts on timber fender faces are always countersunk. It is important to provide sufficient clearance for timber abrasion and wear between the top of the bolt and the timber face. These recesses should always be plugged with timber discs or filled with pitch or mastic to protect the steel components from corrosion. Oversized and empty hardware holes are ideal access ports for insects and marine borers. These areas should be closely inspected to ensure the piles are not being bored into.

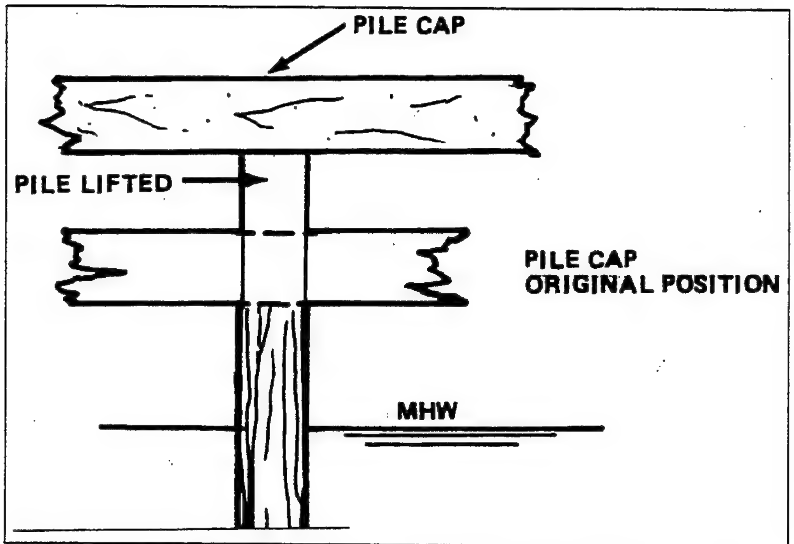



Figure 9-6. One potential consequence of steel connecting hardware failure in timber Construction.

**9.2.6 Abrasion Damage**. Abrasion from suspended sand or silt and from ice during winter months will continually decrease the diameter of piles, as shown in Figure 9-7, unless some means of protecting the piles is used. The rate at which the pile is destroyed by abrasion depends on the amount of debris in the harbor, whether or not there is ice in the harbor, the activity of marine borers, and the velocity of the water moving past the pile.

Timber fender faces are subject to constant abrasion while a ship is in berth. The constant ranging of the vessel fore and aft and up and down will, in time, wear away the outer timber fibers, tending to expose the connecting hardware to contact with the vessel.

Attack by woodgribbles accelerates the rate of destruction of a pile by rendering surface fibers susceptible to removal by abrasion. Abrasion can usually be distinguished from woodgribble attack because abrasion is usually concentrated on one side of the pile while woodgribble destruction is uniformly distributed around the pile. Also, abrasion usually leaves the surface fibers of timber piles rough and protruding from the surface of the sound timber.

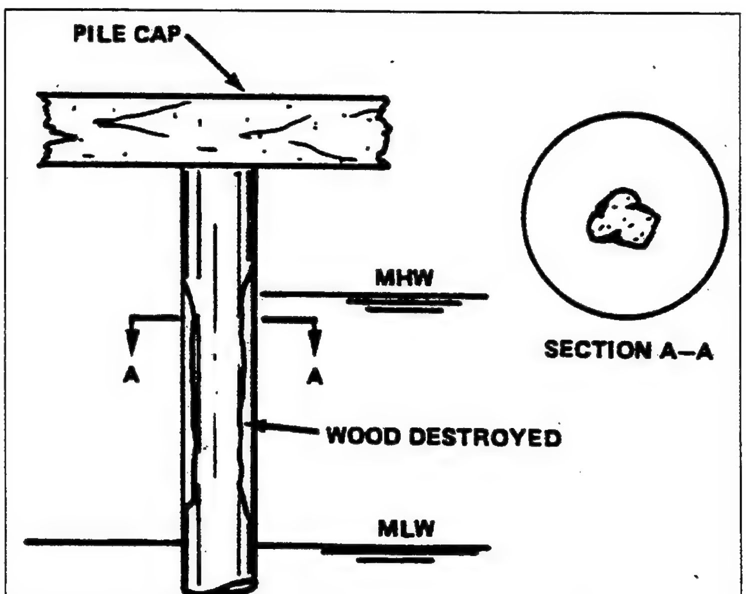



Figure 9-7. Reduction in pile diameter due to abrasion.

## 9.3 Typical Inspection Procedure

**9.3.1 Surface Inspections.** A thorough inspection of all timber structures and their attachments above water should occur. Include annual load testing of the pier decking if heavy equipment or vehicles are to be driven onto the pier. The inspector should be alert, specifically in the areas of stringers, pile caps and top of piles, for signs of discoloration and softening of the wood, accompanied by a fluffy or cotton appearance. This may be an early sign of fungi damage. More advanced deterioration may take on the appearance of fruiting bodies, such as mushrooms. Further down the pile, the inspector should look for burrows or hollows in the wood, surface trenches in the outer layers of the pile, and loss of pile diameter. This may be evidence of marine borer attack.

**9.3.2 Underwater Inspections.** Underwater inspection of a timber waterfront structure should proceed as outlined in Table 9-1.

Table 9-1. Timber Structure Underwater Inspection Checklist

| Checkpoint | Description |
|---|---|
| 1 | Start at the splash/tidal zones.<br>**Note**: A Level I inspection should be done first to identify areas of mechanical damage, repair, and new construction. |
| 2 | Clear a section of the structure of all marine growth and visually inspect if for surface deterioration. Do spot locations rather than cleaning entire structure. |
| 3 | Sound the cleaned area with a hammer and carefully probe with a thin-pointed tool, such as an ice pick. |
| 4 | Descend down the pile, sounding the structure with a hammer wherever there is minimal marine growth, as well as probing carefully with an ice pick. |
| 5 | At the bottom, note and record the depth of the water. |
| 6 | Record visual observations, such as presence of marine borers, losses of cross-sectional area, organism-caused deterioration, location and extent of damage, alignment problems, and condition of fastenings. Use calipers and scales as required. |
| 7 | Where internal damage from marine borers is suspected, ultrasonic techniques are available to support the underwater inspection program. The ultrasonic equipment is only available as a contractor service at this time. |
| 8 | After finishing the underwork, return to the surface and immediately transcribe all observation data into the inspection log. |
| Exposed Area Under Pier or Along Wharf or Dolphin Assembly | |
| 9 | Check wood stringers, pile caps, bearing, and batter and fender piles for missing or broken members. Check dolphins for broken, worn, or corroded cables and cable connectors; and corroded, loose, broken, or missing wedge block, chafing strips and bands, or chock bolt hangers. |
| 10 | Visually examine piling for rot, fungi, and marine borer damage. |
| 11 | Sound the pile areas with a hammer and carefully probe with a thin-pointed tool such as an ice pick. |
| 12 | If an area is in question, take a small boring for laboratory analysis using an increment borer. Once the core is extracted, seal the hole with a creosote-treated plug to prevent easy access of borers to the interior of the pile.<br>**NOTE**: An engineer should be present whenever underwater inspections are made to explain to the diver exactly what he should look for: number and size of piles, type and depth of bulkheads, location of tiebacks, and cross bracing. .The engineer shall evaluate the diver's observations and determine the degree of hazard. |

### 9.4 Equipment and Tools Required

To ensure a thorough inspection, the area must be cleared of all marine growth. This can be done using a "Barnacle Buster" or other types of high-pressure waterblasters. However, when using this equipment, great care must be exercised to prevent damage to the preservative-treated layer of timber.

- Clean small areas with wire brushes and scrapers.

- Sounding of the structure can be performed using a 3-pound sledge hammer.

- An ice pick or pick hammer is required for probing and an increment borer is required if cores are to be taken.

- Timber element dimensions can be checked using a ruler or tape measure.

- A simple fabricated or purchased caliper, is very effective for measuring the diameter of piles.

Inspection data can be recorded underwater using a Plexiglas slate with a grease pencil. Permanent documentation can be achieved through the use of underwater photography, either still photo or television.

## 10.0 STONE MASONRY STRUCTURES

### 10.1 Types of Stone Masonry Structures

Although very few waterfront structures built today are constructed from stone masonry, it is still necessary to be familiar with the inspection of this type of structure. Throughout the 19th century, stone masonry was generally used in constructing graving docks, bridge piers, quaywalls, and wharves. Typically, the quarried stone used was granite set into lime mortar or portland cement mortar.

### 10.2 Deterioration of Stone Masonry Structures

Stone masonry structures typically develop problems at the joints between pieces of stone. Failures of these types of structures usually occur as a result of washout of the joints. In addition, increased earth or hydrostatic pressure causes joints to crack and stones to fall out. Scouring at the base of the structure because of wave and current action and loss of fill from behind the structure are two common types of damage that can lead to serious structural failure.

### 10.3 Typical Inspection Procedure

Stone masonry retaining walls, such as those found on quaywalls and wharves, generally require only a very simple inspection, as follows:

- Begin the inspection at the waterline, checking for excessive weathering and abrasion deterioration, and loss of mortar from the joints.

- Inspect below the waterline, taking note of the general condition of the wall, and paying particular attention to the joints between each stone.

- If there are significant gaps between stones or stones are missing, note the location, depth, and length of missing stone.

- Continue to the bottom of the structure and note any undermining or scouring of the material under the wall structure.

- At any missing stone or undermining, probe the cavity to estimate the extent of the void (if any) behind or below the wall.

- Record the depth of the water at the base of the wall.

- After returning to the surface, immediately transcribe all information into the inspection log if information has not been communicated via hardwire. Also, record in the log the general condition of the wall above the waterline, especially noting all joints from which mortar has washed out.

### 10.4 Equipment and Tools Required

Since the underwater inspection of stone masonry structures involves only a cursory inspection of the joints between stones and the general condition of the wall and its foundation, only a few tools are required. A ruler is used to determine the width and depth of cracks and open joints, as well as the size of missing stones or pieces of stone. It is also useful for quantifying the amount of scouring that has occurred. A length of small-diameter rebar or other suitable probe can be used to check for voids in the fill behind or below the wall. A Plexiglas slate and a grease pencil are used underwater to record any pertinent information, or the information is communicated to topside personnel via hardwire. Small hand tools, such as wire brushes and scrapers, are also useful to clear off cracks and joints.

## 11.0 COASTAL PROTECTION STRUCTURES

Structures designed to reduce the erosive effects of wave action, or to protect harbors from excessive wave action and the formation of sandbars, are classified protection structures. The common coastal protection structures are seawalls, groins, jetties, and breakwaters. NAVFAC Mil-Hdbk-1025/4, "Seawalls, Bulkheads, and Quaywalls" and NAVFAC DM-26.02, "Coastal Protection," provide additional information on the design and configuration of coastal protection structures.

### 11.1 Seawalls

Seawalls are massive coastal structures built along the shoreline to protect coastal areas from erosion caused by waves and flooding during heavy seas. Seawalls are constructed of a

variety of materials including rubble-mounds, granite masonry, or reinforced concrete elements. They are usually supplemented by steel or concrete sheet pile driven into the soil and are strengthened by wales and brace-type piles. Figure 11-1 shows three seawall configurations.

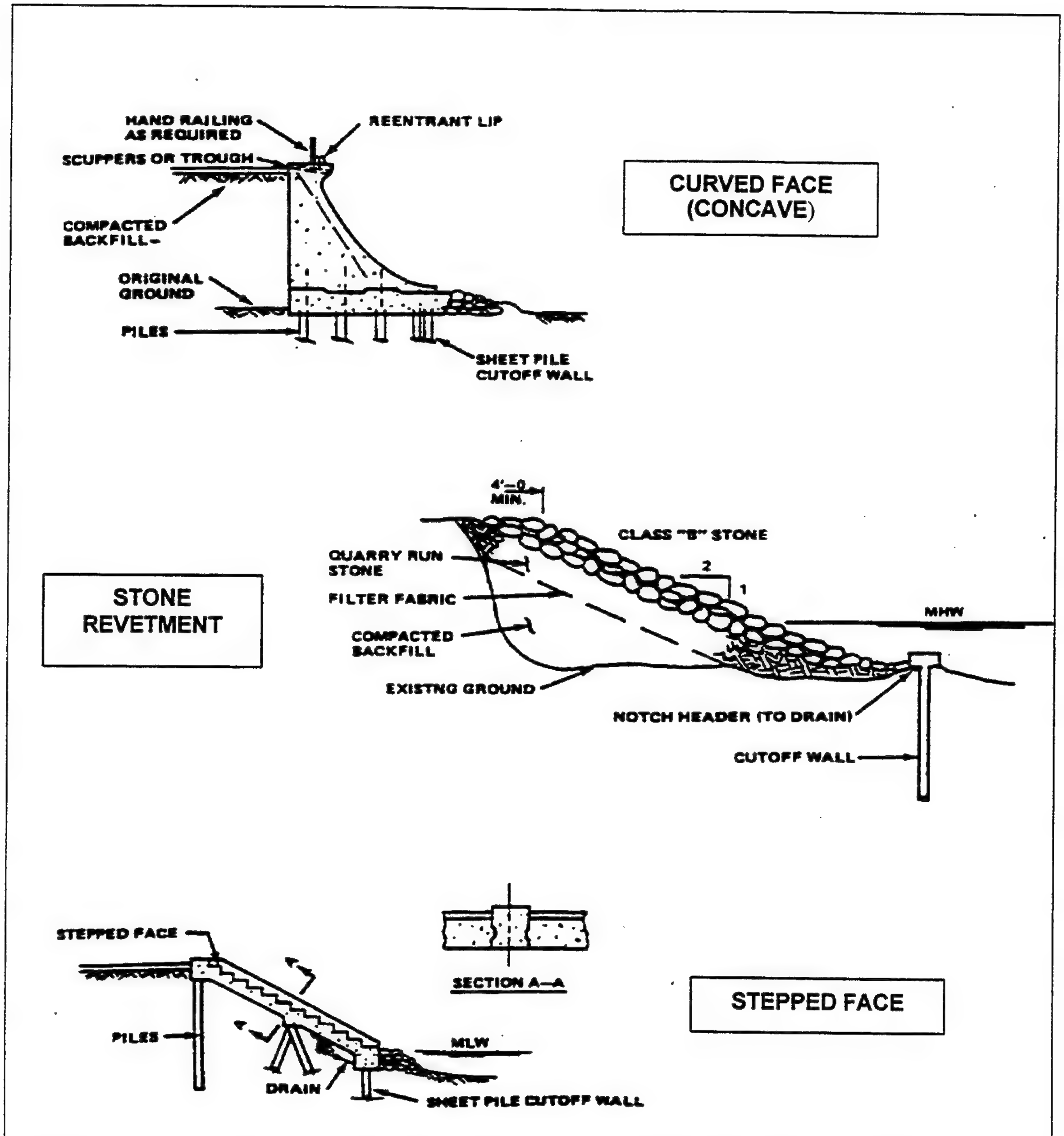



Figure 11-1. Curved Face, Stone Revetment, and Stepped Face seawall configurations.

## 11.2 Groins

Groins are structures designed to control the rate of shifting sand by influencing offshore currents and waves so that erosion of the shoreline is minimized. Groins project outward, perpendicular to the shoreline, and are constructed of large rocks, precast concrete units, reinforced or prestressed concrete piles, steel sheet piles, or timber cribbing filled with rock.

## 11.3 Jetties

Jetties are structures that extend from the shore into deeper water to prevent the formation of sandbars and to direct and confine the flow of water due to currents and tides. These structures are normally located at the entrance to a harbor or a river estuary. Jetties are usually constructed of mounds of large rubble to a height several feet above the high tide mark.

## 11.4 Breakwaters

Breakwaters are large rubble-mound structures located outside of a harbor, anchorage, or coastline to protect the inner waters and shoreline from the effects of heavy seas. These barriers help to ensure safe mooring, operating, loading, or unloading of ships within the harbor. Breakwaters may be connected to the shore or detached from the shore. There are three general types of breakwaters, depending on the type of exposed face. The exposed face may be vertical, partly vertical, and partly inclined, or inclined.

## 11.5 Rubble-Mound Structures

Rubble-mound structures (Figure 11-2) are constructed on the seabed by dumping stones of various sizes from scows and barges until the mound emerges a certain distance above mean sea level. The outer layers of the mound are covered with armor consisting either of large stone or precast concrete units of a number of possible shapes. Rubble is irregularly shaped rough stones, ranging in size up to 1,000 ft3 each and weigh up to 90 tons each. Cobble, also used in rubble-mound structures, is rounded gravel or gravel fragments between 2-1/2 and 10 inches in diameter. Rubble-mound structures are used extensively, chiefly because they are adaptable to almost any depth of water in the vicinity of harbors and can be repaired readily.

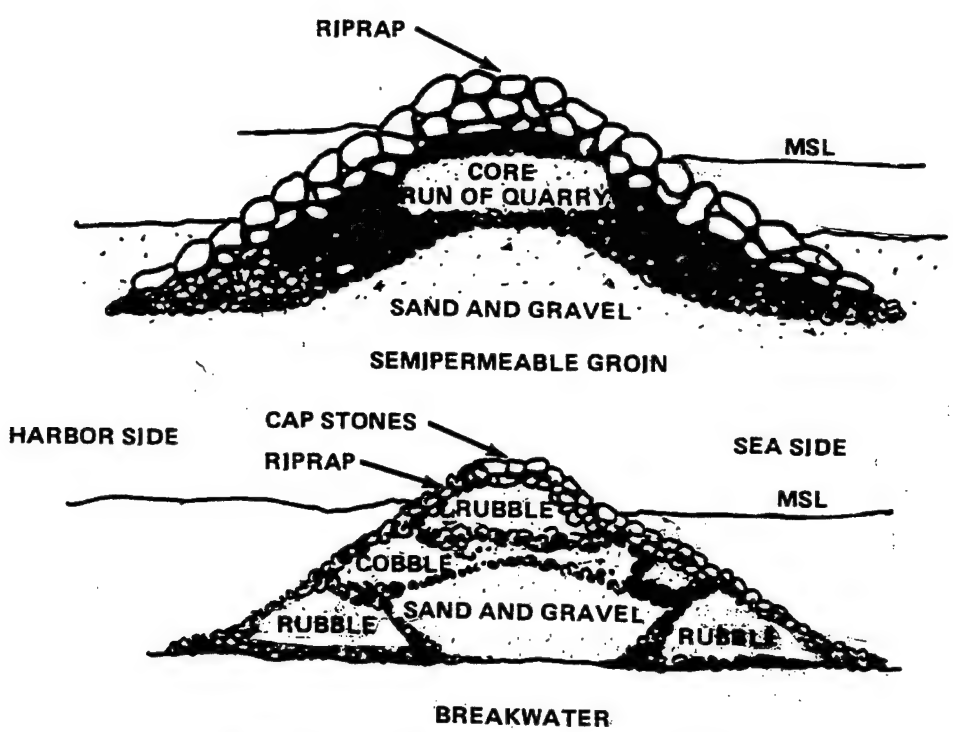



Figure 11-2. Rubble Mound Structures.

## 11.6 Deterioration of Rubble-Mound Structures

The four principal types of deterioration in rubble-mound waterfront structures are:

- Sloughing of side slope
- Slippage of base material as a result of scour by currents
- Dislodgment of stones by wave action
- Excessive settlement of the seabed supporting the structure

During the inspection of seawalls, breakwaters, groins, and jetties, similar to those shown in Figure 11-2, the inspector should check for horizontal and vertical alignment. He should also be particularly watchful for signs of breakage or displacement of large stones or concrete armor elements, and washing out of substrate under the larger stones or concrete elements, particularly at the toe of the structure. These losses can be early signs of eminent structural failures if corrective action is not taken.

Inspection of rubble-mound structures should include:

- Erosion of core material by wave action.
- Erosion of small stones in riprap.
- Stability of armor stones or blocks.
- Breakage and displacement of concrete armor elements.
- Washing out of substrate at the toe of the structures.
- Undermining of foundation.
- High water mark; overtopping.
- Settling of structures.

## 11.7 Typical Inspection Procedure

Inspection of a rubble-mound structure should proceed as outlined in Table 11-1.

Table 11-1. Rubble-Mound Structure Surface and Underwater Inspection Checklist.

| Checklist | Description |
|---|---|
| 1 | Swim around the base of the structure looking for beginning weaknesses in the base, such as washout of small stones and core material. |
| 2 | Note signs of detrimental wave action, such as scouring and sloughing. |
| 3 | Record all pertinent information on a Plexiglas slate. After returning to the surface, transfer the information into the inspection log. |
| 4 | Record the result of the above-water inspection, include a description of the alignment and general condition of the mound, such as dislodgement of stones, gaps, and other weaknesses. |

### 11.8 Equipment and Tools Required

Inspecting rubble-mound structures requires that divers be equipped with recording devices, such as a Plexiglas slate, grease pencil, and cameras.

## 12.0 SYNTHETIC MATERIALS AND COMPONENTS

Inspection of synthetic materials and components is subdivided into the following three categories:

### 12.1 Structural Members

Structural members should be inspected annually when the regular pier inspection is accomplished. The inspection is intended to detect and document:

(1) Cracked, worn, brittle or deformed plastic railings, stanchions, gratings, light standards, or piping; loose or damaged fittings and connections; and exposed fiberglass.

(2) Cracked, worn, or deformed rubber resilient fender components, and/or loose or damaged fittings and connections.

Basic inspection procedures are the same as those outlined for timber or concrete structures.

## 12.2 Coatings, Patches and Jackets

Coatings, patches, and jackets should be inspected annually, or more frequently, depending upon the failure rate of the application. The objective of the inspection is to detect and document:

(1) Pits, cracks, scars or abrasions in coatings.

(2) Cracked, loose or dislodged epoxy patches.

(3) Punctures, embrittlement, tears, rips, or abrasions in fabric, or unlocking of fabric seams in pile jackets.

Basic inspection procedures are the same as those outlined for timber, concrete and steel structures.

## 12.3 Foam-Filled Fenders

Inspection will be done by walking the pier and by use of a small boat. Inspection of foam-filled fenders should be performed more frequently than normal pier inspections and should cover:

(1) Condition of the fender-to-pier connection hardware. Check for operability and signs of corrosion. Check to ensure that the fender is constrained horizontally so that it contacts the bearing surface for its full length. Ensure that the fender is free to float with the tide vertically and rotate around its long axis.

(2) Condition of the fender chain and tire net for net fenders. Check to see that the chain is symmetrical on the fender and that the end fittings are in good working order. Ensure that the chains are protected from the ship hull by the tires, and that the net is not loose.

(3) Condition of end fittings on netless fenders. Check to see that the fittings are in good working order, and corrosion is minimal. Check to see that the fender shell is not cracked or separated around end fittings.

(4) Condition of the fender elastomer shell. Check for cuts, tears, and punctures. Record the size and location of damage on a sketch.

(5) Measure or estimate the diameter of the fender at its smallest point to record permanent set.

Record keeping for foam-filled fenders is very important. In this regard, the fenders should be treated as an item of high-cost equipment rather than an appurtenance to a fixed facility. Each fender should have a unique identification number with a history record that includes date of procurement, manufacturer, date of installation or when fender was put into service, and berth location if permanently installed.

## 13.0 QUAYWALLS

Quaywalls are an integral part of wharves and should be included when other pier components are inspected.

Deterioration of quaywalls is indicated by:

(1) Shifts in horizontal and vertical alignment of sheet piling
(2) Damage or deterioration of the wood, concrete, or steel sheet piling
(3) Wash-out of substrate under the sheet piling, particularly at the toe of the structure.

Item 1 can be detected by visual observation. A complete description of shifts and any apparent cause should be provided. Item 2 is covered by Chapters 6, 8, and 9. Item 3 may be detected by visual inspection in clear water at low tide. If not, then an underwater inspection is required. The following checklist is a useful guide.

Table 13.1 Quaywall Surface And Underwater Inspection Checklist.

| |
|---|
| ☐ Swim around the base of the structure looking for beginning weaknesses, such as washout of small stones and core material. |
| ☐ Note signs of detrimental wave action, such as scouring and sloughing. |
| ☐ Record all pertinent information on a Plexiglas slate and, upon return to the surface, transfer the information into the inspection log. |
| ☐ Record the result of the above-water inspection, which should include a description of the alignment, and general condition of the seawall. |

## 14.0 THE INSPECTION REPORT

For each inspection, a report is prepared. The report includes facility plans with updated descriptions such as size and pile arrangement, an evaluation of the assessed conditions, and recommendations for further action. The report should provide sufficient technical detail to support the assessments and recommendations. Since underwater inspections are specialized, a report format such as the one presented in Table 14-1 is recommended. The recommended format is for each report to first present the Front Information (as defined below) followed by three major sections and the Appendices. This format is used by the NFESC, East Coast Detachment, when conducting underwater inspections and assessments at Naval waterfront facilities.

Table 14-1. Format for Waterfront Inspection Reports.

**Front Information:**

**Report Cover**

↳**Title Page**

↳**Executive Summary**

↳**Executive Summary Table**

↳**Table of Contents**

↳ **List of Figures**

↳ **List of Photographs**

↳ **List of Tables**

**Report Body:**

| | | |
|---|---|---|
| **Section 1** | | Introduction |
| **1.1** | | Background/ Objectives |
| **1.2** | | Inspection Exit Briefing |
| | | |
| **Section 2** | | Activity Description (Information that affects inspection, repair, rate of deterioration, etc.) |
| **2.1** | | Location |
| **2.2** | | Existing Waterfront Facilities at Activity |
| **2.3** | | Waterfront Facilities Inspected |
| | | |
| **Section 3** | | Inspected Facilities |
| **3.1** | | Name of Facility |
| | **3.1.1** | Description of Facility |
| | **3.1.2** | Observed Inspected Condition |
| | **3.1.3** | Structural Condition Assessment |
| | **3.1.4** | Recommendations |
| | | |
| | | Repeat the above as necessary for each facility |
| | | |
| **Appendices** | | |
| **A** | | Key Personnel |
| **B** | | Inspection Procedure/ Level |
| **C** | | Structural Data |
| **D** | | Pertinent Background Information |
| **E** | | Calculations for Structural Assessment |
| **F** | | Backup Data for Cost Estimates |
| **G** | | Cost Estimate Summary |
| **H** | | References |

The objective of the report guideline is to facilitate the writing of comprehensive, standardized, and usable reports. The guideline is the result of many years of experience involving underwater inspections of hundreds of waterfront facilities. The guidelines will assist the inspecting party in preparing the report by pointing out specific information and formats to be incorporated, and by identifying recurrent errors to avoid.

A major objective of underwater inspection report is to provide facility managers with an assessment of the condition of their inspected waterfront facilities. The report shall provide the detailed information needed to substantiate requests for funding to maintain and repair the waterfront facilities. The report shall include the following:

- Identification and description of all major damage and deterioration of the facility.
- Estimate of the extent of minor damage and deterioration.
- Assessment of the general physical condition.
- Recommendations for types of maintenance and repair required.
- Identification of any problems associated with mobilization of equipment, personnel, and materials to accomplish maintenance and repairs.
- Budgetary estimates of costs for recommended maintenance and repairs.
- Estimate of expected life of each facility, with and without recommended repairs.
- Recommendations for types and frequencies of future underwater inspections.
- Updated facility drawings, both hard copy, and electronically-stored versions (which may differ significantly from the drawings available at the activity).
- Documentation of the type and extent (light, moderate, heavy) of marine growth, to help in the planning of future inspections.
- Water depths at each facility.
- Water visibility, tidal range, water current, and any other pertinent environmental conditions.

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